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Paper 1237 WW 2

Journal of the
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CHARACTERISTICS OF NEW TYPE CARGO SHIPS[a]

Douglas C. MacMillan[b]
(Proc. Paper 1237)

SYNOPSIS

It is the intention of this paper to discuss the design characteristics and the functional problems presented by the roll-on/roll-off type of ship, and by certain lift-on/lift-off types having similar basic features, in order that the civil engineer, in his concern with port development, may be as well informed as possible concerning these new types of ships.

Basic Considerations

The object of these new designs is to reduce port time and revise cargo handling methods whereby manual labor by stevedores in terminals and ships is radically reduced or eliminated. Prepacked cargo in freight cars, trailers, van bodies, containers, or pallets may be moved on or off specially designed ships rapidly and with a minimum of manpower. The use of prepacked containers drastically reduces terminal and ship cargo handling time and costs by eliminating the unpacking of the shore side vehicle and re-stowing aboard ship, and repeating at the delivery point.

Basically then, the roll-on/roll-off type vessels which include trainferries, trainships, container ships and certain lift-on/lift-off type vessels handling prepacked cargo containers are characterized by a rapid rate of cargo handling. Various designs in use or under development have rates as indicated in Table 1. The terminal facilities must provide for cargo receipt and dispatch from the ship's side at these rates.

The high rates of loading are also associated with low costs. Analysis of the loading costs of roll-on/roll-off type train ferries indicates costs of two to six cents per measurement ton as compared with $6 to $12 per measurement ton for conventional break-bulk methods.

Note: Discussion open until October 1, 1957. Paper 1237 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 83, No. WW 2, May, 1957.

a. Presented to American Society of Civil Engineers, Pittsburgh, Pa.

b. Pres., George G. Sharp, Inc., New York, N. Y.

Ship Characteristics

The present general cargo ship might be classed as an all-purpose tool, and is usually designed to handle a wide variety of miscellaneous items including bulk cargoes such as ores, grain, coal, special oils, etc., and general freight such as the multitude of manufactured goods exported from this country. The exports carried by one line will include hundreds of commodities in quantities from one to thousands of tons. The deadweights and bale cubic of general cargo ships are represented by those in Table 2.

The bale cubic of these ships varies from 85 to 90 percent of the molded volume of the cargo spaces; in actual practice, about 85 percent of the bale is used for cargo, the difference being due to broken stowage and dunnage. Thus, about 75 percent of the original molded volume is occupied by cargo. On the other hand, roll-on/roll-off and some lift-on/lift-off ships cannot make effective use of as large a percentage of the molded cargo hold volume, and representative data on these types is given in Table 3.

For two ships of identical dimensions and form (although the most efficient designs of each type would not be the same), the general cargo ship will carry roughly twice the volume of cargo per voyage. However, the substantial reduction in port time for the roll-on/roll-off type tends to cancel this deficiency in cargo capacity. Two ships of the C3 size would transport nearly equal measurement tons of cargo per year on a 7,000 mile round trip route, and as the length of the trip decreases, the roll-on/roll-off ship shows to greater advantage. For a 2,000 mile trip coastal run, the roll-on/roll-off trailer ship would carry about 50 percent more annually, and a container ship about 100 percent more annually than the conventional ship. Table 4 presents a comparative estimate for such a case.

Ship's Size and Form

The economics of ship operation results in roll-on/roll-off and container ships having optimum characteristics somewhat different from the conventional general cargo ship. Table 5 presents the dimensions, coefficients and capacities of representative ships of each type.

It will be noted that the roll-on/roll-off types, as compared with the general cargo types, are larger in cubic capacity, smaller in deadweight, finer in form, greater in beam and depth, and less in draft. The finer form and lower port time justify higher speeds than are economically justified for general cargo ships. The speed for the new type general cargo ships tabulated are rather high, which is due to defense requirements.

Structural Design

The requirement for clear decks, without interruption by transverse bulkheads, and large tween deck heights to accommodate vehicles, makes it necessary to design a structure for the roll-on/roll-off ship which differs significantly from that of the standard transversely framed cargo ship. The transverse bulkheads which are carried up beyond the freeboard deck to the uppermost through-running deck in a standard design in order to provide transverse strength must, in the case of the roll-on/roll-off ship, be replaced by deep web frames in conjunction with deep beams closely spaced. Spacing of such webs varies between 8' - 0" and 12' - 0", depending upon the

size of the vessel. With the web frames mandatory, it follows that the spaces between the webs at decks and side shall be carried by longitudinal framing. The deck thicknesses which are required to take the panel load of concentrated wheel loads of heavy vehicles such as trailer trucks, are considerably more than in the standard ship. To keep the increased deck thicknesses to a minimum, the spacing of the longitudinal deck beams (20 to 21 ins.) is reduced from the spacing usually applied to a standard type vessel of the same size (30 to 36 ins. frame spacing).

Because of the heavy deck thicknesses in conjunction with longitudinal beams, and the great depth of ship caused by the deep tween decks, the roll-on/roll-off vessel has longitudinal strength characteristics well in excess of the requirements for a standard type vessel of the same length and number of decks.

An exception to the foregoing would be presented by the construction of one of the oldest types of roll-on/roll-off vessels, i.e., the single deck ferry. Here transverse strength is provided by box columns or vertical girders, arranged port and starboard or on the centerline. These columns or webs are usually continuous and are held at the bottom shell girders and the main deck transversely carrying, as a cantilever, the promenade deck which forms the cover over the vehicles. For this vessel a transverse framing system is the advantageous type of construction.

One of the first roll-on/roll-off vessels for unlimited ocean going service is the MSTS vehicle ship. This vessel was designed to also carry general cargo. This dual requirement, plus the requirement to keep deck heights low, made it necessary to depart from the above described general concept of structure for the roll-on/roll-off vessel, and revert to the transverse framing system. Deck beams were also best located transversely to take the distributed loading of very heavy vehicles such as tracked tanks. Since the beams are in line with the frames (30 ins. spacing) the plating had to be made rather heavy in conjunction with fore and aft panel breakers. Here also the excessive deck thickness has contributed to the improved longitudinal strength of the vessel as compared to the requirements of the standard vessel.

Ship Types

Ships which are presently in use and which attain the high loading rate desired, are the single deck rail car ferries and the well known Seatrains. The single deck car ferries are loaded by the roll-on/roll-off principle, while the freight cars are lifted on and off the Seatrains by a shore crane installation in each port of call. Some of the newer types being considered are briefly described below:

a) Roll-on/Roll-off Sloped Deck Trainship

A trainship design, (Fig. 1) has been developed in an effort to produce a vessel of approximately the same dimensions and capacity as the Seatrain but which required no special facilities on ship or shore for the loading and unloading of the freight cars. This trainship can dock at the usual car float terminals found in harbors all around our coast, and load and discharge its freight cars via these, exactly as is done in car float operation. In order to accomplish this, the cars roll off and on at normal float bridge level or pier level.

It was found that two car decks could be so arranged within the ship that the lower deck would load or discharge at pier level through the vessel's bow and the upper deck similarly through the stern. This meant accenting decks having a slope to the horizontal, a feature which did not present any difficulties since the slopes or gradients required were those acceptable in normal railroad practice. This ship can be designed with essentially flush decks so that it can also carry trailers.

By using this principle, it was possible to design a two-deck Trainship with the same capacity as the Seatrain four-decker, and having approximately the same length, but smaller depth and larger beam. (Fig. 2). The total inside volume of the Trainship is slightly smaller than that of the Seatrain. The underwater portion of the Trainship not utilized for cargo or machinery spaces is small, the vessel being of fairly shallow draft and fine form. At the midship section, the Seatrain and Trainship utilize about the same percentage of their cross section areas for cargo, but in going towards the ends, the Trainship utilizes a progressively greater percentage of the cross section area than does the Seatrain.

A Trainship of the same capacity and speed as the Seatrain would be less expensive to construct due to reduced hull steel, less outfit and equipment, and smaller horsepower. However, it is in the terminals that the greatest saving is effected, since the Trainship may use existing terminal facilities or a relatively inexpensive bridge as compared with the expensive shore crane installation.

b) Roll-on/Roll-off Horizontal Deck Trainship (Fig. 3)

A trainship with either one or two horizontal decks can be loaded from one end by means of an apron hinged on shore. It is preferable to load through the stern, since the breadth of this portion of the ship permits a better arrangement of tracks than could be obtained with bow loading.

The single deck ship typical of railroad ferries and barges, is adaptable to most port conditions. The horizontal double deck vessel presents some complications due to the grade of the loading apron. To minimize the grade, the hinge of the apron on shore should be at mid-height between the two car decks at the average tide level. In this way the variation of tide level can be used to advantage, the lower deck being loaded between mean and high tide, while the upper deck is loaded between mean and low tide. When tidal range is approximately equal to the tween deck height, optimum loading conditions and minimum bridge requirements are attained. This principle is being employed for the new Trainships for Alaska Steamship Company.

c) Roll-on/Roll-off End Loading Sloped Deck Trailership

This type vessel is illustrated in Figure 4. Its method of loading is similar to that of the sloped deck Trainship shown in Figure 1. This ship provides maximum ease and speed for trailer loading since the vans move on or off in an uninterrupted stream. However, it requires special pier facilities consisting of an apron projecting out from the pier so that both ends of the ship can be loaded simultaneously.

d) Roll-on/Roll-off Side and Stern Loading Trailership (Fig. 5)

The Marad design "Turnpike" Class ship is of this type and has three horizontal decks for trailer stowage. It is not as rapid loading as the sloped deck

ship, as the trailers must be maneuvered up and down ramps and turned. This vessel has the great advantage of being adaptable to most existing piers or harbor installations.

e) Roll-on/Roll-off Stern Loading Trailership (Fig. 6)

The Pan Atlantic Steamship Corporation Trailerships are of this type, and have four horizontal decks loaded over the stern by a two-level bridge to the two main trailer decks. Internal ramps are provided to the lowest and highest trailer decks.

f) Container Ship Loaded by Crane (Fig. 7)

At present Pan Atlantic Steamship Corporation is operating two T2 tankers carrying containers above the Weather Deck which are loaded by shore cranes.

The response to these services has been extremely favorable. Additional tankers are being converted, and contract designs are being prepared for major conversions that will handle about 240 - 35-ft. containers but with the cranes installed aboard the ship as discussed later.

Loading Characteristics

As indicated above, roll-on/roll-off type ships may be loaded over the stern, through the bow or through sideports. Of the ship designs now in use, the single deck train ferries load over the stern via a transfer bridge, and the trailerships load through the stern (TMT), and overhead by crane ("Ideal X"). Newer designs in the contract stage follow this pattern. The two-deck trainship for Alaska Steamship Company will load via a single transfer bridge while the trailership for Pan Atlantic Steamship Corporation will load via a two-level ramp. The MSTS Vehicle Ship under construction uses both stern and sideport loading ramps and internal ramps.

Bridge Requirements

The transfer bridges for single deck train ferries in ports with moderate tides (such as the U. S. East Coast and the Great Lakes) use a gantry supported bridge of 70 to 100 ft. in length, which is hinged at the shore end, and supported by the gantry and ship's stern at the outboard end. The dead load is supported by the short pivot and the gantry via counterweights. The live load is supported by the shore pivot and the ship. The transfer bridge is designed so that it will twist, permitting the outboard end to follow the ship as it lists when strings of cars are pushed on to or pulled off the ship. The design conditions of these transfer bridges generally provide for 10 degrees of list to either side, and a maximum grade difference between adjacent cars of 5 percent, or generally maximum grades of plus or minus 5 percent.

Where tides are greater, the bridge must be longer in order not to exceed the 5 percent grade difference limitation between coupled cars. The bridge at Vancouver, British Columbia, which serves a single deck train ferry, and where maximum tides reach 16 ft., is about 200 ft. long, composed of an inner section about 110 ft. long and an outer section about 90 ft. long.

It is, of course, necessary to design the ship and bridge to have matching loading characteristics. The transverse stability characteristics of the ship

can be adjusted in the design stage within limits. Generally a compromise must be reached, since an easy rolling ship is desired for good seagoing characteristics, while a stiff ship is advantageous for loading. For the Alaska trainship, the ship design attains reasonably good sea characteristics, and at the same time restricts the angle of list to 10 degrees when handling a full string of heavy cars on the outboard track. In some designs, the outboard rows must be removed in half strings in order not to exceed the 10 degree limitation, which is dictated by bridge design and freight car coupler characteristics.

As strings of cars are moved on and off the ship, there is also a change in grade of the bridge produced by the change in ship trim, resulting from the shifting weight. There is also a change in grade due to list. These changes in grade may be of the order of two percent and must be considered in the design of the facilities and when loading.

For loading trailerships, the ramp requirements are considerably less exacting due to greater permissible grades, substantially less change in list, and generally lower unit loads. It is also possible to effectively stabilize the ship during loading by shifting water ballast, restricting the trim to plus or minus one percent and the list to plus or minus one degree. While stabilizing a trainship is possible, the ultimate cost is considered to be more than designing the bridge to match the ship.

Bridge Design

The bridge proposed for Alaska Steamship Company at Whittier, Alaska, where the maximum tide is 19 ft. and where the tidal range 50 percent of the time is only 9 ft., is a movable bridge 135 ft. long which can be elevated or lowered at the shore end to connect with either of two track levels. Either track level will serve any track on the ship by means of switches on the bridge and ship. The upper train level on the ship (Main Deck) will be loaded with the bridge at the upper shore level, and the lower train level (Second Deck) with the bridge at the lower shore level. The bridge can serve both decks within the grade limits of 5 percent, for the maximum combinations of tide, ship draft, list and trim during any tidal cycle. The bridge will also be used to load and unload trailers on the upper deck of the ship, from the upper shore level, with an upward grade limit of 15 percent.

This design of bridge was chosen in preference to a two-section pivoted bridge about 250 ft. long of equivalent performance, because it is cheaper and lighter. In effect, a movable bridge span is replaced by fixed shore ramps. The following is a description of the general characteristics and operation of the proposed transfer bridge.

The bridge is designed as a Pratt truss of a depth of 1/10 of the span with 12 panels. The outboard end panel has been specially designed for support by the ship. The top chords of the truss have been extended over the inboard end panel and are slotted to pass a bar hanger, thus giving stability to the top chord at this end. The outboard end of the top chord of the truss will be stabilized by a rigid leg which is pinned through the top chord. Floor beams are pinned under the center of the bottom chord, to allow for a 10 degree list of the vessel. The inboard hinge at the bottom of the truss is formed by a large pin which is extended into a boxlike section. The box contains a buffer spring on either end for shock loading and is to be constructed to slide vertically in guides at the inboard gantry tower. Both sides of the slide are

connected by a rigid box section beam. The bar hanger passing through the slotted top chord is pinned at its upper end to an overhead cross beam, the ends of which are constructed to also form a slide running in the same guides as for the lower section.

The ship end of the bridge is guided onto the vessel by a locating casting on the centerline of the bridge which fits over a horn fastened to the stern of the vessel and is seated on bearing plates located on both sides of the vessel at the stern. The bridge is further guided at the outboard gantry by an overhead cross beam to which the aforementioned rigid leg is pinned. The beam is extended forming a slide, running between guides which are fastened to the gantry tower. The ends of the slides are curved to allow the beam to remain guided while the vessel is listing.

The gantry towers are bolted to the foundations with the inner end post of the inboard gantry also rigidly bolted to the abutment which has been carried beyond the upper level to absorb the bridge end reaction and shock loading via the tower structure.

The top of the foundation for the inboard gantry tower has been carried to a level to clear the lower slide. The top of the foundation for the outboard gantry has been carried sufficiently high to form a positive step for the vessel in light condition at high tide in conjunction with a buffer arrangement connected to the face of the ship end of the foundation.

The deadload of the bridge is counterbalanced at the inboard gantry by weights running inside the tower; the weights are connected to 4 cables which run over large sheaves with the other end of the cables connected to equalizer beams. The live load, plus the remainder of deadload is carried at this point by a screw arrangement supported by girders at the top of the towers. The screw raises and lowers the bridge between the two levels.

At the outboard gantry the deadload of the bridge is also counterbalanced by a weight running inside of the towers. The bridge is raised and lowered at this point by an auxiliary counterweight which is lowered on to the main counterweights when raising the bridge, and lifted well clear when the bridge is supported at the ship's stern.

Internal bracing for wind loads will be provided either under the bridge in the form of tension struts or in the form of a wide face plate on one chord of the trusses.

The gap between the bridge floor system and the ship's deck, will be covered by an auxiliary hinged bridge plate which will automatically position itself as the bridge rests on the deck. The ship must be moved forward to clear the outboard end of the bridge when the bridge is moved up or down.

Cranes

New designs of container ships with a large number of large containers stowed below the weather deck will require shipboard mounted traveling gantry cranes for loading. Synchronized independent suspension of the ends of long containers is essential for rapid and accurate placement of the containers. New designs for shipboard application are required. These cranes should have a lifting capacity of about 25 long tons, a hoisting rate of about 75 ft. per minute, a traversing rate of 125 ft. per minute, and should be capable of moving fore and aft with light load, and should be powered from 440 volts, 3 phase, 60 cycle a-c current. It is necessary for the traversing track

which extends over the ship's side to be retractable in order not to interfere with ship maneuvering and docking.

Trends

Whereas the roll-on/roll-off type ship has made considerable progress in the past few years, and some new services are being initiated, a trend is discernible to large capacity heavy lift ships of the lift-on/lift-off type. These designs are being considered for both domestic and overseas services and will be featured by high loading rates and quick turnarounds.

TABLE I. LOADING RATES

TYPE SHIP	CARGO HANDLING RATE MEASUREMENT TONS PER HOUR
Single Deck Train Ferry	4,000
Seatrains	1,700
Two-Deck Trainship	4,000
Trailership	6,000
Containership (Lift-on/Lift-off)	1,200 to 3,000
Pallet (Sideport - fork truck)	350 to 700
Conventional Ship	20 to 40 per gang 150 to 500 total
Vehicle Ship	2,000 approx
Self Unloading Bulk Carrier	4,000 to 8,000

TABLE 2. GENERAL CARGO SHIPS

TYPE	SPEED Knots	TOTAL DEADWEIGHT Tons	CARGO DEADWEIGHT Tons (a)	BALE CAPACITY Cu.Ft.	STOWAGE FACTOR Cu.Ft. per Ton (a)	(b)
Ships Presently in Use						
Liberty	10.0	10,800	9,900	500,000	50.5	52.8
C1B	14.0	9,120	8,150	437,000	53.6	64.3
C2-S-B1	15.5	9,280	8,050	546,000	67.8	77.3
C3-S-A2	16.5	12,100	10,500	658,000	62.6	68.3
Victory AP3	16.5	10,700	9,430	453,000	48.0	60.8
Mariner	20.0	13,400	11,640	737,000	63.3	75.0
Navy Stores Issue	18.5	7,220	5,440	358,000	65.7	75.7
Proposed New Maritime Designs						
Freedom	16.0	8,800	7,840	497,000	65.8	75.0
Clipper	18.0	10,900	9,730	600,000	61.2	74.4
Seafarer	18.0	14,940	13,550	744,500	56.0	72.3

(a) Cargo deadweights based on 6,000 miles range.
(b) At designed range.

TABLE 3. ROLL-ON/ROLL-OFF AND CONTAINER SHIPS

(WEIGHTS IN LONG TONS; VOLUMES IN CUBIC FEET)

Type	Speed Knots	Total Dwt.	Cargo Deadweight(a) (b)	(c)	Cargo Capacity Inside Container (d)	(e)	Bale Cap. Inside Ship	Cargo Space Molded Vol.	Ratio (f)
Single Deck Train Ferry	17.5	3,850	2,350	1,440	160,000		495,000	582,000	.275
Two Deck Trainship	18.0	8,540	8,000	4,560	576,000	480,000	1,314,000	1,546,000	.372
Trailership	18.0	8,710	7,225	5,075	600,000	483,000	1,453,000	1,710,000	.350
Containership	15.5	5,640	4,930	3,730	503,000	415,000	678,000	798,000	.63
Vehicle Ship - Design Vehicle Load	18.0	6,500	5,950	.....	548,000(g)	.....	(h)	900,000	.61

NOTES:
(a) Cargo deadweight based on 2,000 mile range.
(b) Including weight of container.
(c) Excluding weight of container.
(d) Including open deck cargo.
(e) Excluding open deck cargo.
(f) Ratio, cargo inside containers to molded volume of cargo space.
(g) Cube of vehicles, approximate.
(h) RO/RO holds = 630,000; General Cargo holds = 136,500. Total bale = 766,500.

TABLE 4. SUMMARY OF STUDIES OF A C-3 VESSEL ON A 2000 MILE ROUND TRIP COASTAL ROUTE

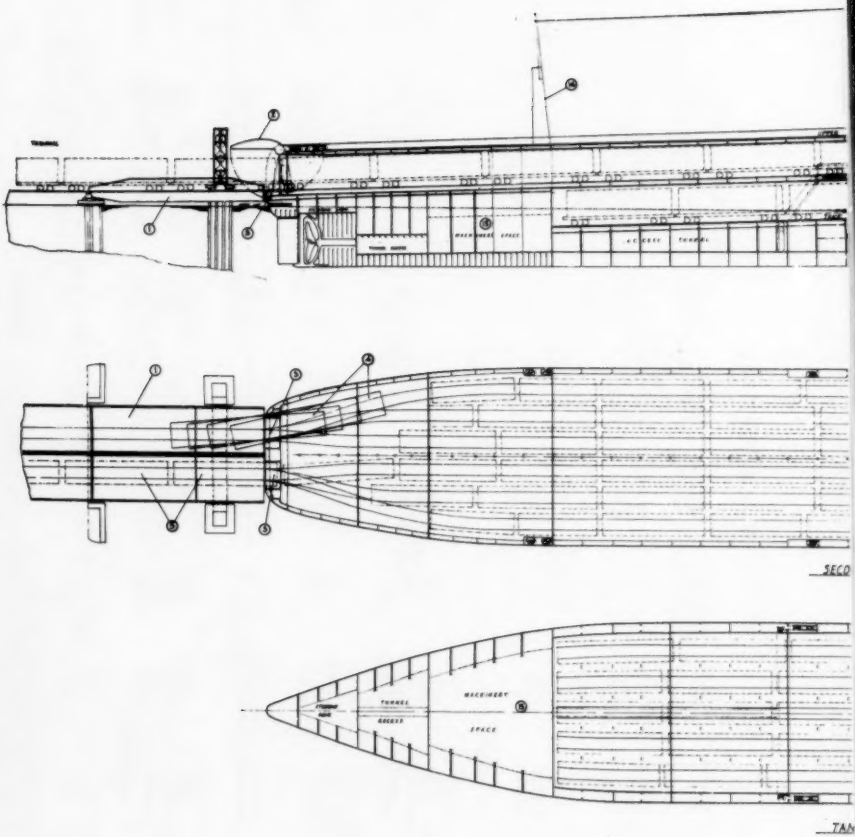
Type of Ship	Conventional Ship		Trailer Ship		Container Ship	
1. Volumes, Cu. Ft. and Tonnage						
Molded Cargo Space	770,000		770,000		770,000	
Usable Cargo Space	578,000		260,000		420,000	
Percent Used	75		34		55	
Measurement Tons Used	14,450		6,500		10,500	
2. Round Trip Port Time - Days						
Loaded both ways	20		2		3	
Loaded only one way	10		2		3	
3. Round Trips per Year						
Loaded both ways	13.9		46.5		40.2	
Loaded only one way	23.0		46.5		40.2	
4. Measurement Tons of Cargo per Year						
Loaded both ways	402,000		605,000		844,000	
Loaded only one way	332,000		303,000		422,000	
5. Cargo Transportation Cost *	$/ton	%	$/ton	%	$/ton	%
Loaded both ways						
Ship Operating and Fixed	3.85	18.5	2.60	71.5	1.98	32.1
Trailer, Container & Tractor	-	-	.91	25.2	.23	3.7
Cargo Handling	17.00	81.5	.12	3.3	3.96	64.3
Total	20.85	100.0	3.63	100.0	6.17	100.0
6. Cargo Transportation Cost *						
Loaded only one way						
Ship Operating and Fixed	4.67	21.5	5.20	71.3	3.96	46.1
Trailer, Container & Tractor	-	-	1.81	25.0	.46	5.4
Cargo Handling	17.00	78.5	.24	3.3	4.16	48.5
Total	21.67	100.0	7.25	100.0	8.58	100.0

* Terminal to Terminal

TABLE 5. COMPARATIVE CHARACTERISTICS

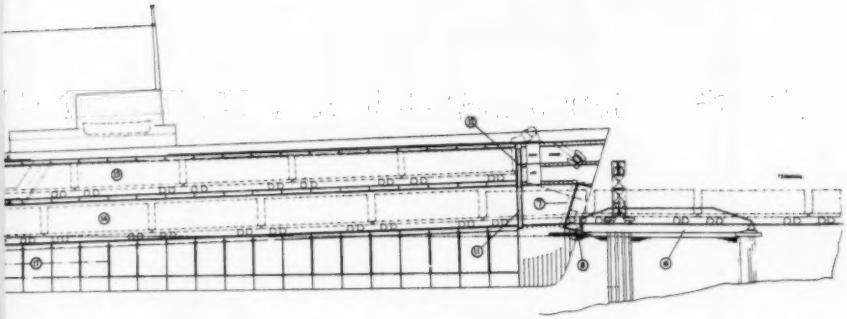
	CONTAINER SHIP	TRAILER SHIP	RO/RO PALLET	TRAIN-SHIP	VEHICLE SHIP	General Cargo			
						PRIVATE DESIGN	CLIPPER	SEAFARER	MARINER
Dimensions, feet									
Length Overall	508.7	637.5	320.0	579.0	499.0	499.0	496.0	530.3	563.6
Length between Perpendiculars	485.0	591.0	296.0	541.0	462.0	460.0	460.0	494.0	528.0
Length on Waterline	.....	619.0	304.0	565.0	479.0	474.0	465.0	.....	535.5
Beam at Load Waterline	72.0	83.2	50.0	86.5	78.0	70.0	73.0	74.5	76.0
Beam, Maximum, feet	72.0	87.0	50.0	86.5	78.0	70.0	73.0	74.5	76.0
Depth to Upper Deck	.....	62.0	.....	67.8	48.8	.....	.....	.....	.....
Depth to Main Deck	40.5	46.0	30.5	49.1	34.3	42.5	41.3	44.5	44.5
Depth to 2nd Deck	.....	30.0	16.75	30.5	22.3	34.0	32.6	35.5	35.5
Design Full Load Draft	21.8	22.5	14.0	20.0	22.0	29.0	28.0	31.5	29.75
Maximum Draft	24.0	23.0	14.0	22.3	27.0	29.5	30.0	.....	31.5
Coefficients Based on LWL and Maximum Draft									
Block	.608	.573	.509	.59	.60	.685	.619	.685	.624
Prismatic	.684	.632	.567	.64	.62	.698	.630	.697	.635
Midship Section	.889	.922	.897	.92	.969	.982	.983	.983	.983
Waterplane	.712	.792	.754	.79	.737	.785	.731	.770	.745
Speed Length Ratio	.705	.733	.930	.743	.838	.77	.838	.809	.870
Beam Draft Ratio	3.30	3.69	3.57	3.93	3.55	2.41	2.61	2.36	2.55
Displacement & Tonnage, tons									
Designed Full Load Displacement	13,216	18,560	3,050	15,300	14,100	18,190	16,900	21,800	21,100
Total Deadweight	8,021	8,710	1,650	6,630	6,500	12,500	10,900	14,940	13,410
Gross Tonnage	.....	14,600	700	9,500	.....	10,400	10,350	11,500	9,215
Capacities, cubic feet									
Bale Cubic	678,000	1,453,000	150,000	1,314,000	766,500(d)	662,000	601,000	744,500	737,000
Cargo Cubic	503,000(a)	600,000(a)	135,000	576,000(a)	.....	563,000(b)	511,000(b)	631,000(b)	626,000(b)
Speed and Power									
Design Sea Speed, knots	15.5	18.25	16	18	18	16.5	18	18	20
Normal Shaft Horsepower	6,000	13,600	3,300	11,500	12,000	8,500	11,000	12,500	17,500
Number of Propellers	1	2	2	2	2	2	1	1	1
Normal Cruising Range, miles	3,900	3,900	2,000	2,500	13,000	21,500	15,000	16,000	19,000
Subdivision and Stability									
Standard of Subdivision (c)	1	1	1	2	2	1	1	1	1
Metacentric Height in Full Load Departure Condition, feet	3.6	4.5	3.7	3.3	4.1	2.6	3.4	4.7	4.5

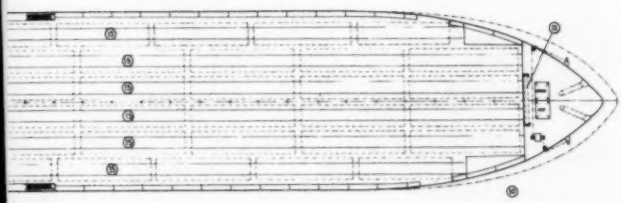
NOTES:
(a) Inside containers.
(b) Assuming 15% loss for dunnage and broken stowage.
(c) Number of adjacent compartments that can be flooded without loss of ship.
(d) 630,000 for RO/RO loading.



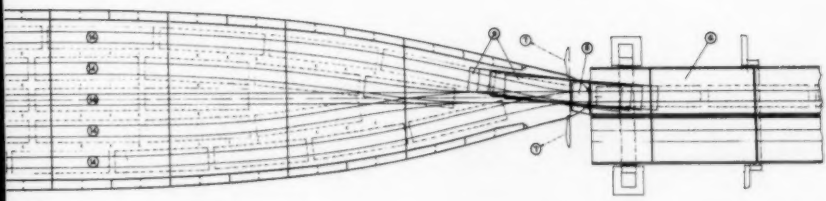
ROLL ON ROLL OFF TRAIN SHIP
END LOADING SLOPED DECKS
482' x 76' x 50'-9" D x 26' d

FIGURE. 1



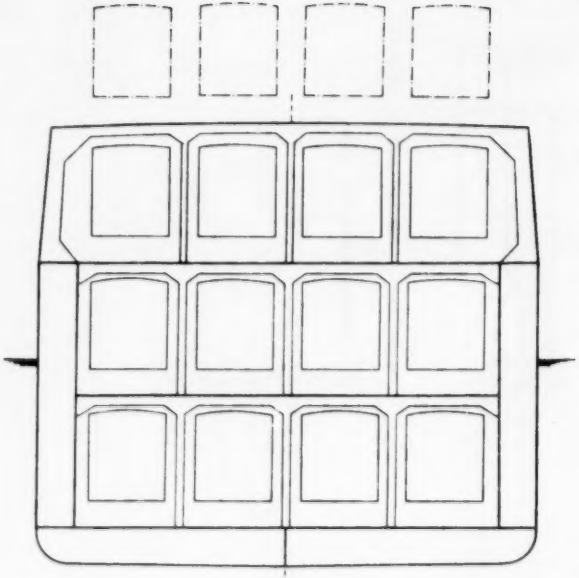


D DECK



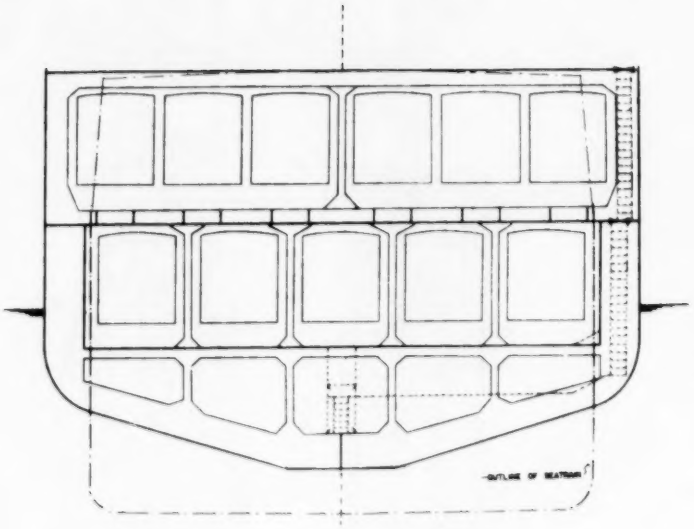
TOP

FREIGHT CAR CAPACITY - 100 - 45' CARS
OR
TRAILER CAPACITY 180



468'-0" L x 63'-6" B x 59'-3" D

MIDSHIP SECTION OF EXISTING SEATRAIN TYPE VESSEL

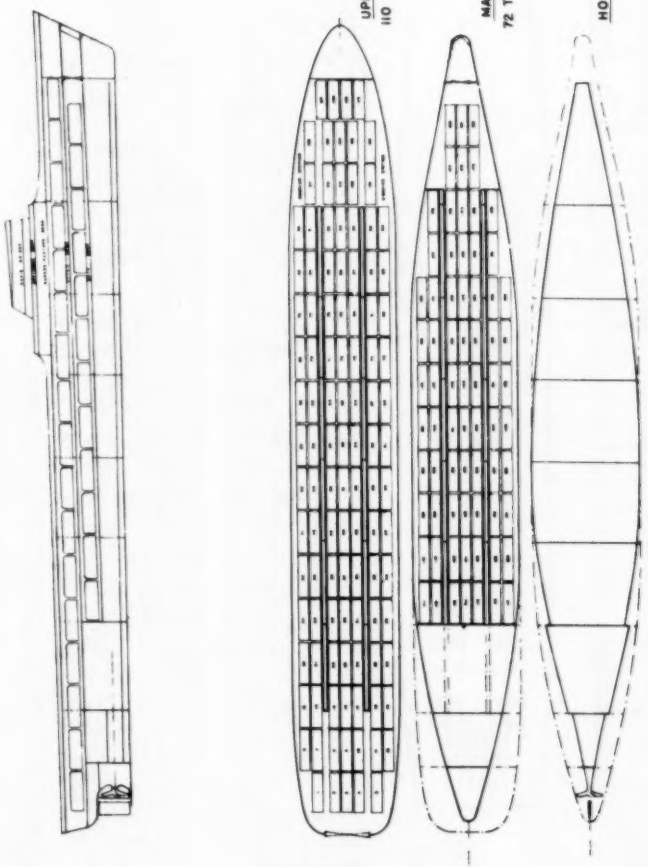


420'-0" L x 78'-0" B x 50'-9" D

MIDSHIP SECTION OF TRAINSHIP
SHOWING SUPERIMPOSED OUTLINE OF EXISTING SEATRAIN TYPE VESSELS

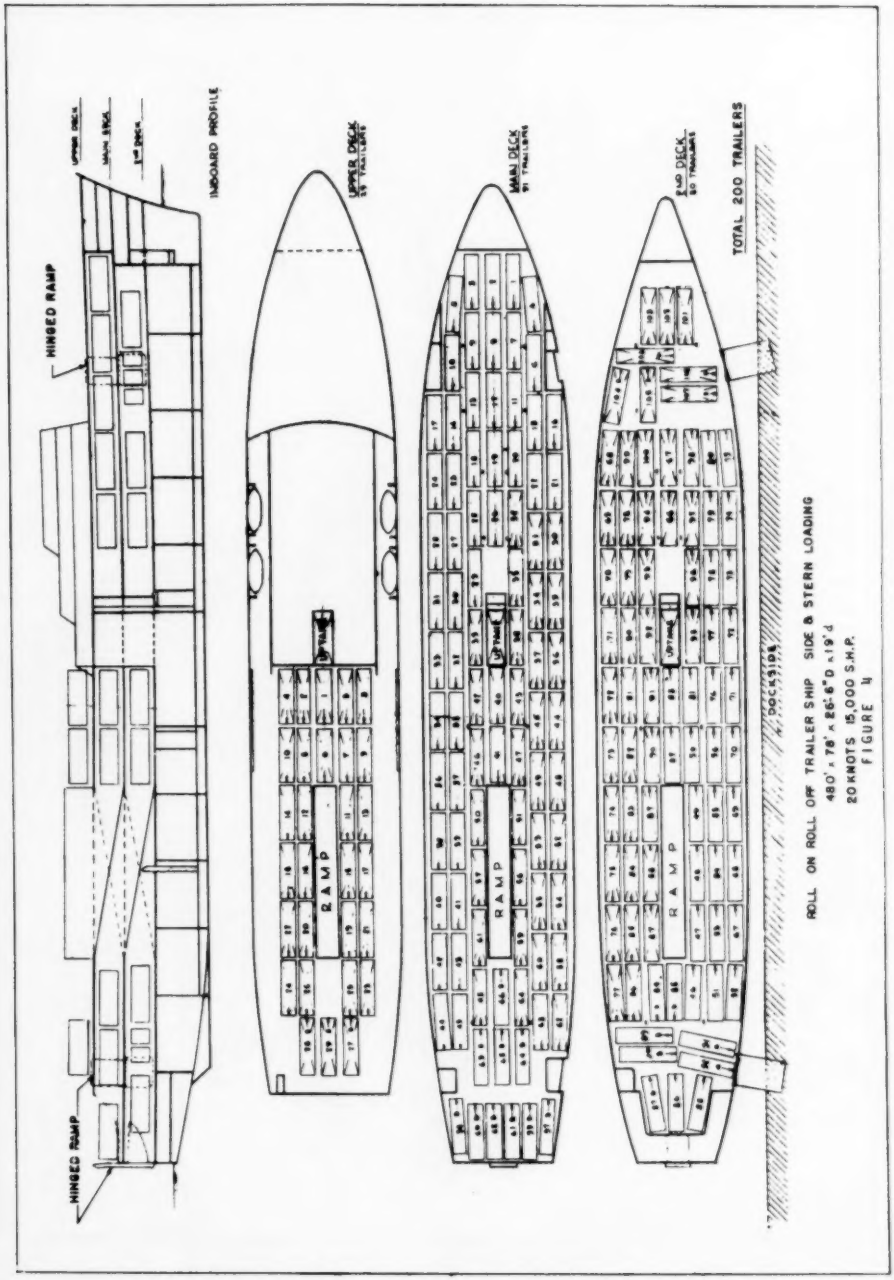
COMPARATIVE MIDSHIP SECTIONS
ROLL ON ROLL OFF TRAINSHIP & SEATRAIN

FIGURE 2

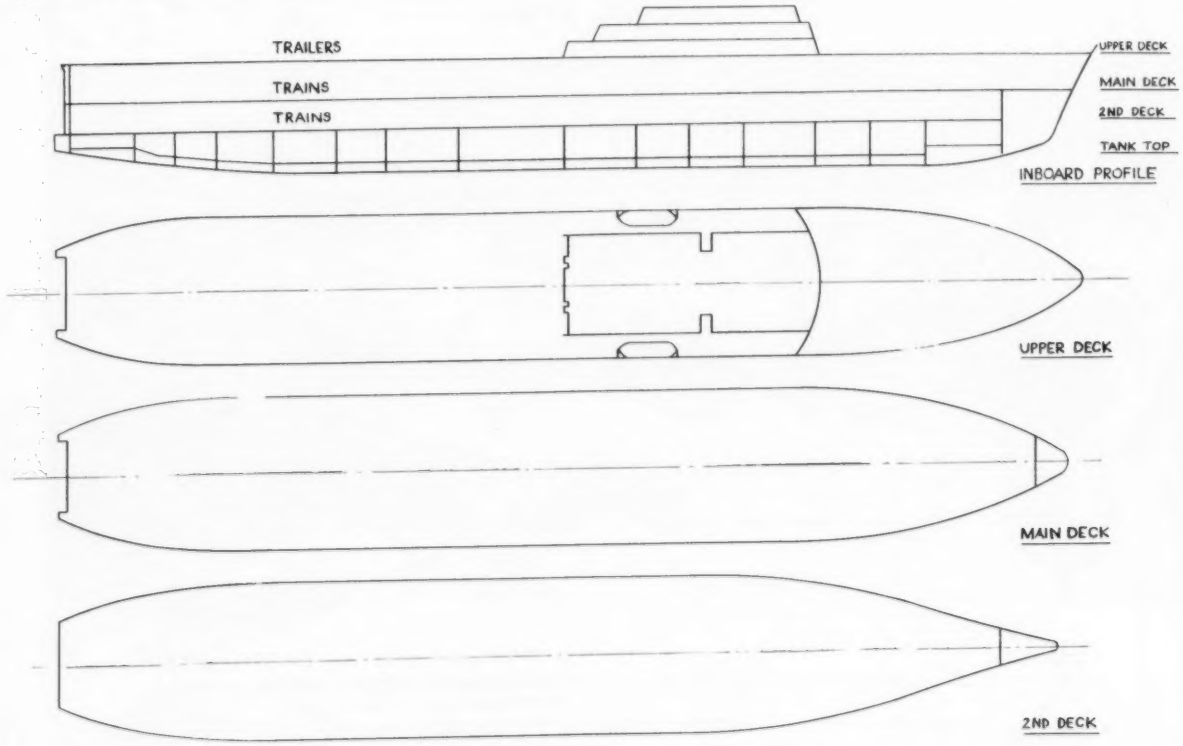


ROLL ON ROLL OFF TRAILER SHP
END LOADING - SLOPED DECKS
555' x 76' x 34'D x 20'd
20KNOTS 12,500 SHP

FIGURE 3



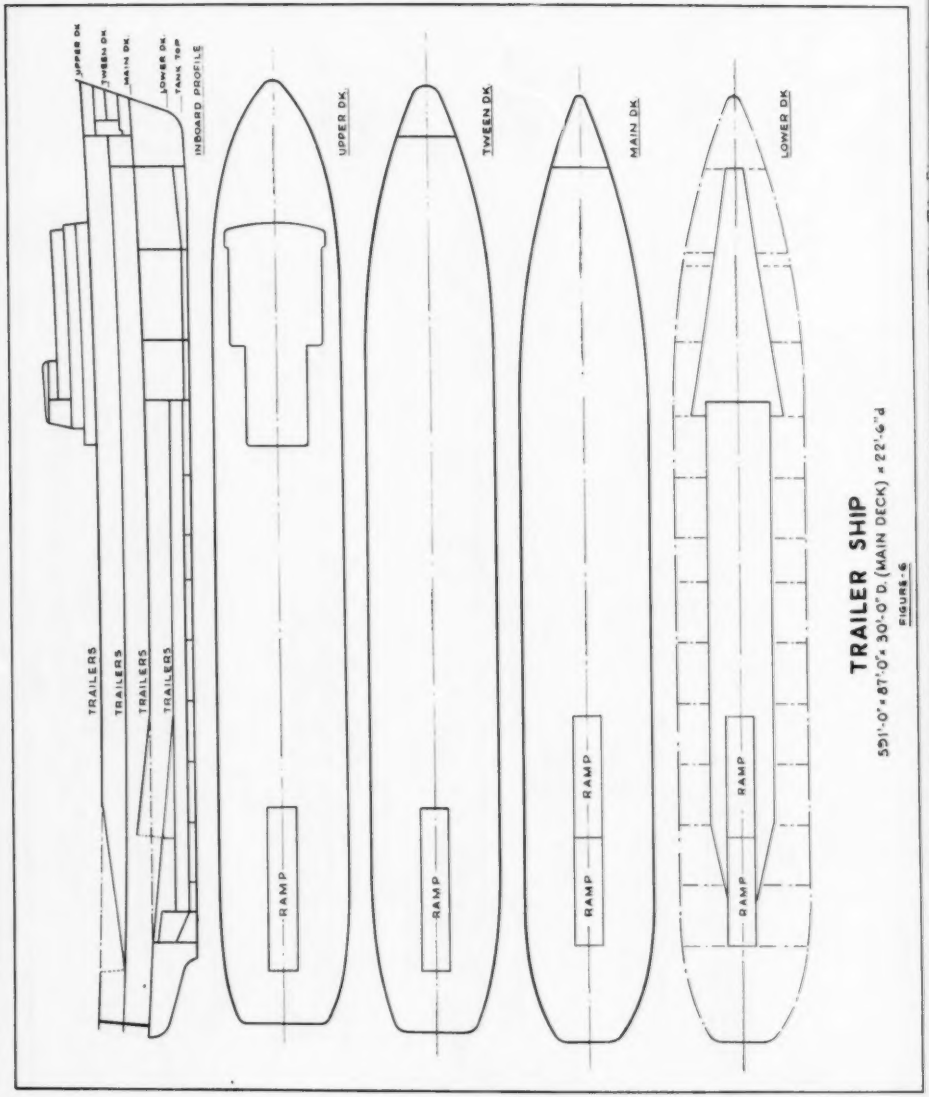
ROLL ON ROLL OFF TRAILER SHIP SIDE & STERN LOADING
480' x 78' x 26'-6" D x 19'-4"
20 KNOTS 15,000 S.H.P.
FIGURE 4



TRAIN SHIP

539'-0" x 86'-6" x 67'-10½" D (UPPER DECK) x 22'-6" d

FIGURE 5



TRAILER SHIP

591'-0" x 87'-0" x 30'-0" D. (MAIN DECK) x 22'-6" d

FIGURE-6

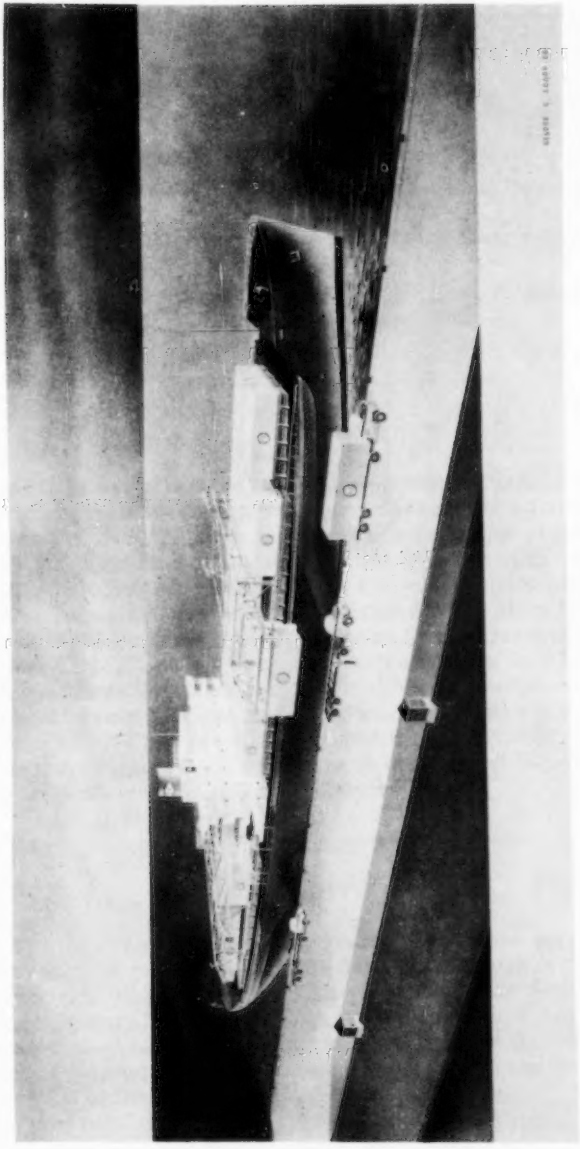


FIGURE 7

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

HURRICANE DESIGN WAVE PRACTICES[1]

Charles L. Bretschneider,[2] A.M. ASCE
(Proc. Paper 1238)

ABSTRACT

Methods are presented which allow a better estimate than heretofore for obtaining hurricane design waves for offshore and coastal structures. It is intended that this paper supplement the information presented in an earlier Technical Report on Surface Waves and Offshore Structures by REID and BRETSCHNEIDER, 1953 (reference 1 at end of this paper).

Two important concepts are introduced which allow one to obtain with fair reliability the hurricane design wave and its recurrence interval. First, the "Energy Index" concept originally proposed by R. O. REID(2) is used to obtain the design hurricane for deep water wave conditions; and second, techniques are employed for computing wind waves over a shallow bottom taking bottom friction into account as proposed by BRETSCHNEIDER.(3) To illustrate these techniques, a hurricane is selected off the coast of North Carolina, for which wave computations are made.

INTRODUCTION

The hurricane design wave problem has become very important the past few years, in regards to both the design of offshore structures and coastal structures. Unless one can predict accurately the design hurricane wave and its recurrence interval, there is little assurance, if any, of absolute safety, or a justified economic balance between cost and the safety of life and property. Computations of design wave forces can be no more accurate than the selection of the design wave.

Note: Discussion open until October 1, 1957. Paper 1238 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 83, No. WW 2, May, 1957.

1. Presented at the American Society of Civil Engineers, 1956 Annual Convention, October 15-19, 1956, Pittsburgh, Pa.
2. Hydr. Engr. (Research), Beach Erosion Board, Corps of Engineers, U.S. Dept. of the Army, Washington, D.C.

Originally, design wave practices made use of wave forecasting relationships based on constant wind speed and direction. This is not a correct procedure for hurricane design, where the wind over the fetch varies both in direction and speed. In bringing the waves across the continental shelf, original practices assumed only refraction and shoaling changed the wave height. Later studies revealed that bottom friction has considerable effect on modifying the wave height in shallow water, prior to reaching the critical breaking depth of 1.28 H_B, where H_B = breaker height. In regards to earlier practices there existed two main drawbacks once a design hurricane was selected. First, an average wind speed and fetch length were selected, from which the hurricane design wave was predicted. This was purely subjective, and as many hurricane design waves could be obtained as there were forecasters making the forecast. Second, no allowance was made in shallow water to account for wave energy loss due to bottom friction, and the final selection of the hurricane design wave frequently had little meaning over a good guess.

It was apparent that the above practices were not very satisfactory. Nevertheless, structures had to be built using at that time the best available information on hurricane waves; hoping either that a hurricane would never be experienced, or that one would be experienced as a test before too many structures were built.

Because earlier techniques were unsatisfactory, new approaches were made to the subject of hurricane wave forecasting, and these have been used during the past few years. The methods were improved and revised as time went on, and to date the present material represents the latest proposed method for obtaining the hurricane design wave. Further improvements are still in progress, and refinements can be made as one learns more about hurricanes and their behavior, and as one obtains more wave observations within hurricanes, both in deep and shallow water.

Generation of Deep Water Wind Waves

Before discussing generation of deep water waves under hurricane conditions, it is necessary first to review the deep water wave forecasting relationships developed for constant wind speed and direction. At present there are two schools of thought in wave forecasting, the SVERDNUP-MUNK-BRETSCHNEIDER[4,5] Method and the PIERSON-NEUMANN-JAMES[6] Method. The first method makes use of the term significant wave and the second, the wave spectrum. The wave height frequency distribution and the wave period frequency distribution may be obtained from relationships given by PUTZ[7] using the significant wave height and period, respectively. The wave spectrum method makes use of the theoretical frequency distribution of wave heights given by LONGUET-HIGGINS.[8] The results by the two methods for wind waves should give comparable results since the LONGUET-HIGGINS theoretical distribution is almost identical to that of PUTZ[7] which is based on actual wave data. However, agreement is not always reached, and it is beyond the scope of this paper to go into detail about all reasons for disagreement, except that the major difficulty apparently arises primarily because the two methods do not make use of the same wave data for describing wave generation, and also the data in each case have not been analyzed by the same techniques.

As far as wind wave generation is concerned, the most important wave data existing is given by JOHNSON[9] obtained from Abbotts Lagoon, California. In regards to these data, waves were measured (not visually observed), wind and wave instruments were carefully calibrated, and the records carefully analyzed by reproducible techniques. The results are tied in with the PUTZ[7] frequency distribution, and therefore to the best of the authors knowledge represent the best available data that can be used for predicting wind waves having similar dimensionless values of gF/U^2, gH_o/U^2 and $gT_s/2\pi U$. The range in values of gF/U^2 for the Abbotts Lagoon data (Figure 1) extends through the range in gF/U^2 for high wind speed and short to moderate fetches for ocean waves. There may be some argument as to a possible scale factor effect involved in the use of these data. However, any error here must certainly be less than that which one would encounter from the use of actual ocean deep water wave data, where the limits of fetch length, wind duration and wind speed are difficult to measure, and where wave observations are purely subjective.

It is noted in Figure 1 that the curve of $gT_s/2\pi U$ vs gF/U^2 over the range of the Abbotts Lagoon Data[9] has a slope of 1/4, and the curve of gH_o/U^2 vs gF/U^2 has a slope of 1/2, which over the range shown are almost the same as the curves revised by BRETSCHNEIDER.[5] It is debatable which give a better fit. However, the present interpretation results in better agreement with observations than the relationship given in reference 5. Wave tank data for the far left of the graph and ocean data from relatively low wind speeds and very long fetches for the far right of the graph are omitted, since these data fall outside the range of most hurricane wave generation. Furthermore, it is convenient in the development following to use the simple slopes of 1/4 and 1/2 respectively for wave height and period. Below the range of $gH/U^2 = .235$, one obtains the following formulae:

$$1)\quad H_o = .0555\sqrt{U^2F}$$

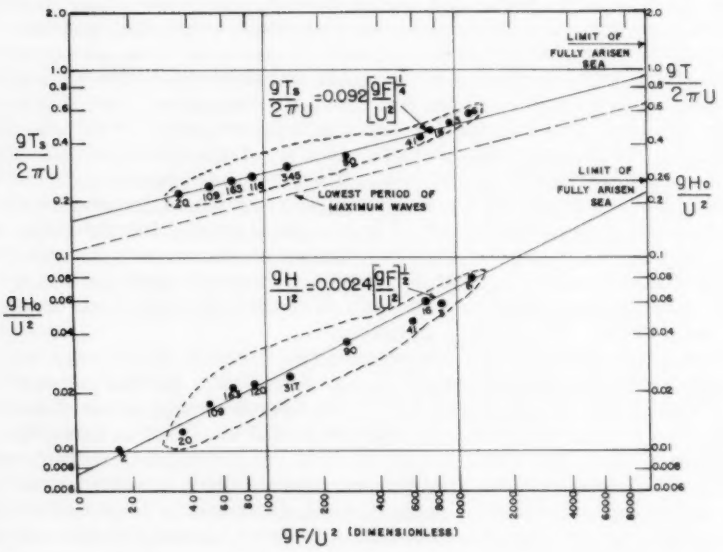
$$2)\quad T_s = .5\sqrt[4]{U^2F}$$

where

H_o = significant deep water wave height, feet
T_s = significant deep water wave period, seconds
U = wind speed, knots (constant)
F = fetch length nautical miles

It is interesting to note that use of these two formulae over the generation range of gF/U^2 shown in Figure 1 gives a constant ratio of $H/T^2 = 0.22$. This can be interpreted as an apparent steepness of 1 to 23, which agrees very closely with that of 1 to 20 used in design for many years. However, as far as the maximum wave is concerned, a greater steepness would result.

The minimum duration t_{min} (in hours) to generate waves of H_o and T_s can be obtained from:



NOTES:

1. Source of data Abbotts Lagoon (J. W. Johnson)

2. No. beside points represents no. of points averaged

3. Symbol ⬭ indicates limit of scatter of data

4. $H = 0.0555 \sqrt{U^2 F}$, $T = 0.50 \sqrt[4]{U^2 F}$ } for U = knots

F = nautical miles $\begin{cases} H \geq 0.023 U^2 \\ T \geq 0.45 U \\ F \geq F \text{ min.} \end{cases}$

FIGURE 1. GENERATION OF DEEP WATER WIND WAVES FOR SHORT FETCHES AND HIGH WIND SPEEDS

4) $C_g = 1.515\ T_s$, group velocity (knots)

This results in

5) $\frac{F_{min}}{t_{min}} = .57\sqrt[4]{U^2 F_{min}} = 1.14\ T_s$

If one is interested in the maximum wave, which is higher than the significant wave, it is sometimes desirable to predict the lowest possible period of the maximum wave. In deep water $H_{max} = 1.87\ H_o$ (PUTZ) and the critical steepness from wave theory results in $H/T = 0.877$, from which one obtains:

6) $T^1 = 1.46\sqrt{H_o} = .69\ T_s$

That is, the design wave period could be either (or both) T^1 or T_S depending on type of structure in mind.

Figure 2 represents forecasting relationships based on the above equation, and are intended primarily for short fetches and high wind speeds, i.e., for $gF/U^2 \lesssim 10^4$. This is equivalent to about 85 to 90 percent of a fully arisen sea, defined as $gH_o/U^2 = .26$ the upper limit for wave generation under any particular wind speed, assuming unlimited fetch length and wind duration. It can be seen from Figures 1 and 2 that it takes much longer time and fetch length to reach 100 percent of a fully arisen sea, than it does for the first 85 to 90 percent. This is very useful to keep in mind when discussing hurricane wind wave generation.

The above forecasting relationships can be used for hurricane wave generation in a manner similar to that of WILSON[10], wherein use is made of the space-time wind field pattern. This would give most of the wave information for hurricane wave generation, more than is required for design purposes. Since the space-time wind field distribution requires very much work, a simple method is developed for obtaining directly the highest waves that can be generated by a hurricane. Reference 10 might be used as a check.

Generation of Deep Water Wind Waves under Hurricane Wind Conditions

Three of the most important fundamental differences between generation of wind waves under hurricane conditions and that of normal wind waves are as follows:

(a) Winds within a hurricane are not constant in speed.
(b) Winds within a hurricane are circular in direction as opposed to straight line.
(c) The hurricane moves over waves generated at various angles of direction to the path of the storm. This fact may cause pyramidal waves formed by two wave trains approaching at a large angle to each other.

(a) and (b) can be easily accounted for by replacing U^2F in equations 1 through 6 by:

$$\int_{x_1}^{x_2} U_x{}^2 dx,$$

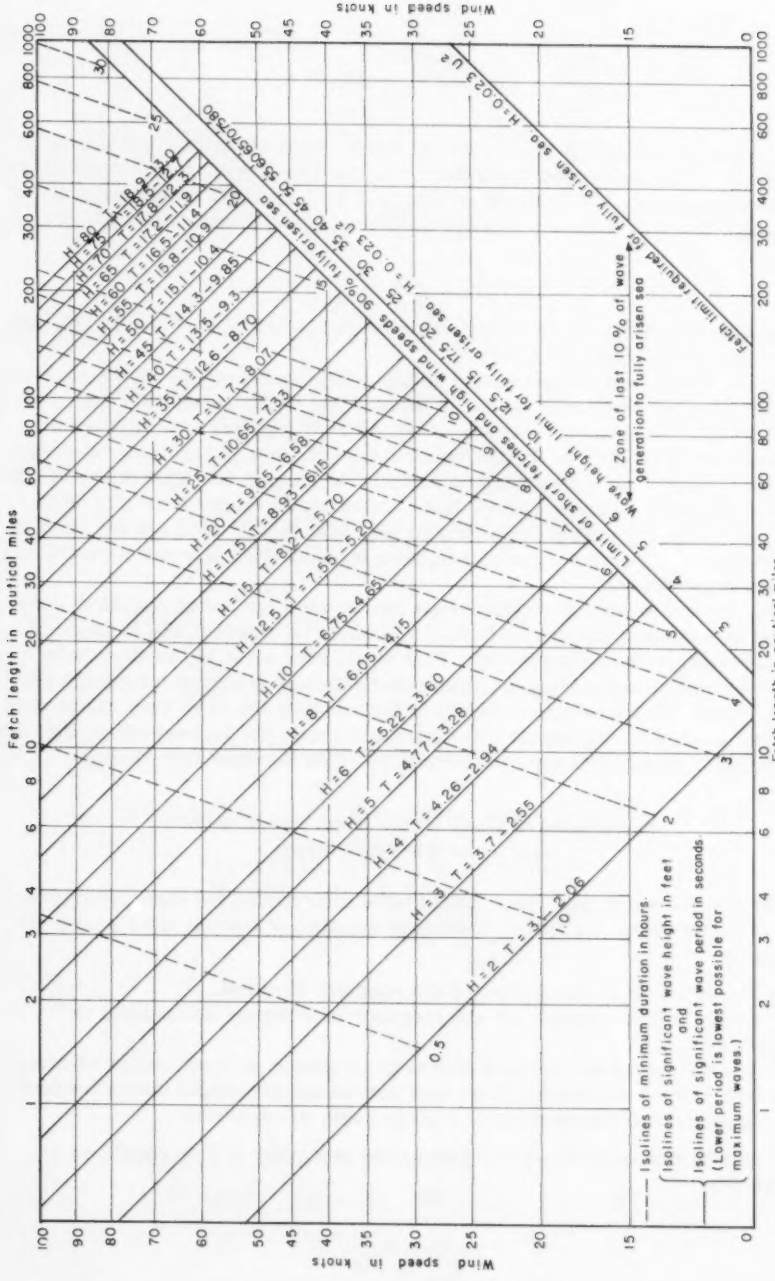

Wind speed in knots
Fetch length in nautical miles
Wave height limit for fully arisen sea
Zone of last 10 % of wave generation to fully arisen sea
Fetch limit required for fully arisen sea
Limit of short fetches and high wind speeds
90% fully arisen sea
Isolines of minimum duration in hours.
Isolines of significant wave height in feet and
Isolines of significant wave period in seconds. (Lower period is lowest possible for maximum waves.)

FIGURE 2. DEEP WATER WAVE FORECASTING CURVES FOR SHORT FETCHES AND HIGH WIND SPEED

where U_x is the component of wind in some arbitrary straight line direction. In case of a moving hurricane the space-time distribution of $(U_x^2)_t$ can be used. The pyramidal wave of item (c) above can best be considered by statistical means or from the critical steepness of $H/L = 1/7$ or $H/T^2 = .877$.

In order to by-pass the tedious work involved using the space-time wind field distribution, use is made of certain inherent characteristics of hurricanes, the most important of which are:

(a) R, radius of maximum wind, and
(b) Δp, atmospheric pressure reduction at center from normal pressure.

Other factors to consider in obtaining the surface wind are:

(a) f, coriolis parameter
(b) frictional drag or shear stress between the surface wind and the surfaces (either water or land) (See Figures 4 and 5)
(c) V_F, forward speed of storm

The above five factors are used to obtain the surface wind, a result of cyclostrophic and geostrophic flow, reduced in speed and deflected toward the center of the storm due to frictional drag. Cyclostrophic flow represents a balance between the centrifugal force and the pressure gradient force. This is the primary balance near the center of the storm. Geostrophic flow represents a balance between coriolis force and the pressure gradient force. This is the primary balance near the outer periphery of the storm. Within the zone of maximum wave generation both cyclostrophic and geostrophic flow are important. The equation for cyclostrophic wind is:

$$7) \quad \frac{V_c^2}{r} = \frac{1}{\rho}\frac{dp}{dr} = \frac{1}{\rho}\frac{\Delta p}{r}\frac{R}{r}e^{-\frac{R}{r}}$$

The equation for gradient wind, which includes both geostrophic and cyclostrophic flow is given by:

$$8) \quad \frac{V_g^2}{r} + fV_g = \frac{1}{\rho}\frac{dp}{dr} = \frac{1}{\rho}\frac{\Delta p}{r}\frac{R}{r}e^{-\frac{R}{r}}$$

where

V_c = cyclostrophic wind speed
R = radial distance to maximum wind
r = radial distance to V_c
dp/dr = pressure gradient
ρ = density of air
V_g = geostrophic wind
f = coriolis parameter = $2\omega \sin \phi$
Δp = reduction in central pressure from normal

Figure 3 is the solution of equation 8, where the units are in terms of nautical miles, knots, and hours, and where the atmospheric pressure anomally Δp is in inches of mercury. The coriolis parameter in radians per hour for various latitudes are given below:

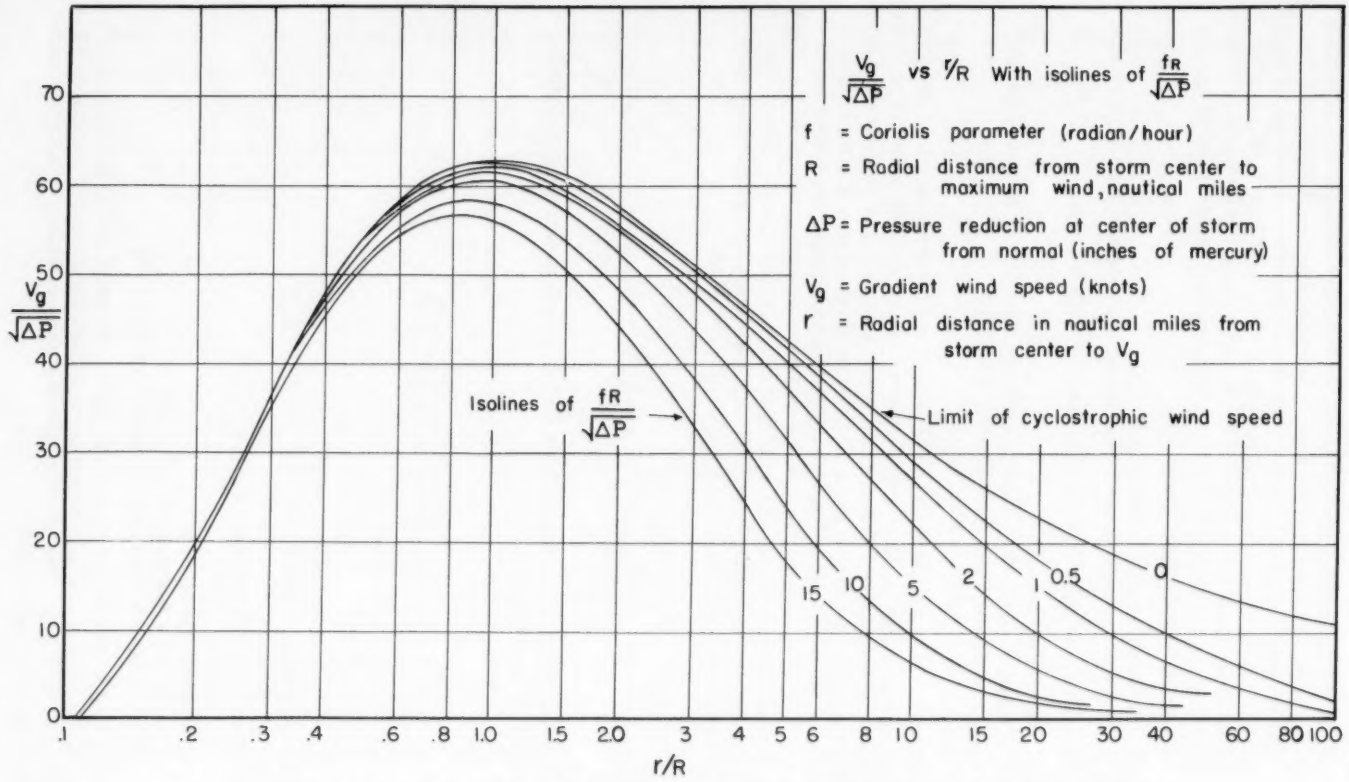

$\frac{V_g}{\sqrt{\Delta P}}$ vs r/R With isolines of $\frac{fR}{\sqrt{\Delta P}}$
f = Coriolis parameter (radian/hour)
R = Radial distance from storm center to maximum wind, nautical miles
ΔP = Pressure reduction at center of storm from normal (inches of mercury)
V_g = Gradient wind speed (knots)
r = Radial distance in nautical miles from storm center to V_g
Isolines of $\frac{fR}{\sqrt{\Delta P}}$
Limit of cyclostrophic wind speed
0
0.5
1
2
5
10
15
$\frac{V_g}{\sqrt{\Delta P}}$
r/R
0 10 20 30 40 50 60 70
.1 .2 .3 .4 .6 .8 1.0 1.5 2.0 3 4 5 6 8 10 15 20 30 40 60 80 100

FIGURE 3 SOLUTION OF GRADIENT WIND EQUATION FOR HURRICANES

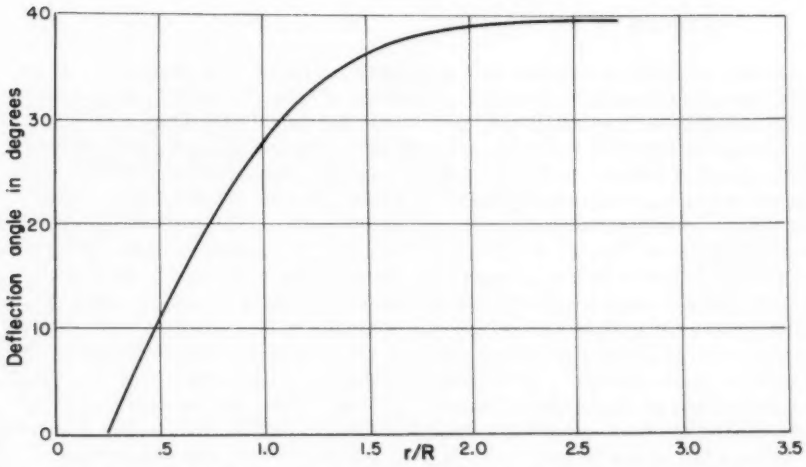


FIGURE 4 WIND DEFLECTION AS A FUNCTION OF r/R
(After U.S. Weather Bureau)

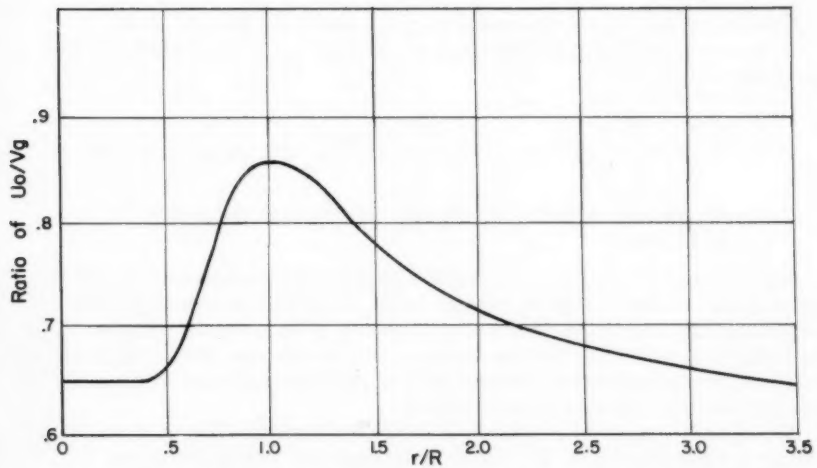


FIGURE 5 RATIO OF SURFACE WIND SPEED TO GRADIENT WIND SPEED AS A FUNCTION OF r/R
(After U.S. Weather Bureau)

Lat (degrees)	25	30	35	40
f (hours^{-1})	.222	.264	.300	.338

It now becomes necessary to make three corrections to obtain the actual surface wind speeds; (a) change in direction of velocity vector due to surface drag, (b) change in speed of velocity vector due to surface drag, and (c) change in speed of velocity vector to take into account the effect of forward speed of storm. Items (a) and (b) may be obtained from empirical data. Based on one hurricane analyzed in detail by the U.S. Weather Bureau and MYERS,(11) such relationships are developed, and are reproduced in dimensionless form in Figures 4 and 5 respectively. In regards to item (b) the present practice is to superimpose the forward speed of the storm on the resulting surface wind field obtained by use of Figures 3, 4, and 5. This is permissable for a slow moving storm, but some error will result for rapidly moving storms. Actually some value less than that of a fast moving storm should be used. However, little data is available at present to justify such a reduction, and as far as determining the design wave, the hurricane will be moving at a forward speed slow enough so that it will be sufficiently accurate to add the full speed of the storm to the wind field of the stationary storm.

Using Figures 3, 4, and 5, and Δp, and R for Hurricane Hazel of October 15, 1954, the surface wind field for the stationary storm was computed. This is shown in Figure 6. Upper concentric circles represent a plan view of isolines of constant wind speed for the stationary storm. The lower curves show cross sections of the velocity distribution along a line located at a distance R from the center, where R is the radial distance to maximum wind. The symmetrical curve is for the absolute value of the velocity vector $|U_o|$, and the other represents the component U_x along the line X' - X'.

In terms of wave generation along the line X' - X', one must use the value of:

$$\int_{X_1}^{X_2} U_x^2 dx,$$

where the limits of X_1 and X_2 have arbitrarily been set at U_x = 15 knots.

Since a model hurricane is readily defined by its parameters R and Δp, use is made of the concept of Energy Index, originally proposed by REID(1) for classifying hurricanes and their frequency of occurrence. Figure 7 shows the relationship of $R\Delta p$ versus frequency of occurrence of 69 U.S. hurricanes. The recurrence interval of storms for any particular section of coast for design purposes is indeed a separate study.

The kinetic energy of a hurricane is proportional to $R\Delta p$, and the wave energy is proportional to H^2, which from Equation 1 is proportional to U^2F. This leads to the following definition of Energy Index for hurricane wind wave generation:

$$9) \quad \int_{X_1}^{X_2} U_x^2 dx = U_r^2 F = \text{function of } R\Delta p,$$

where U_r is the root mean square of the wind speed component U_x, obtained by use of equation 12.

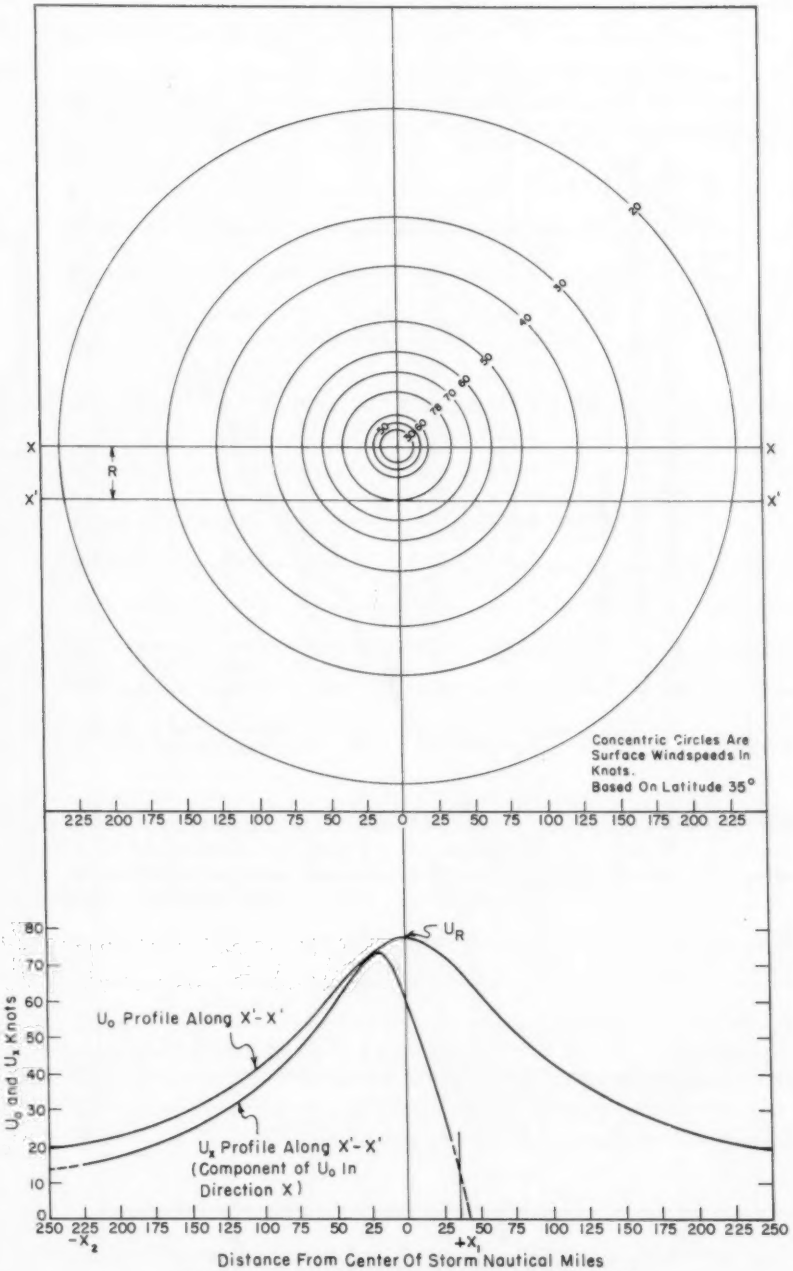

X
X'
R
Concentric Circles Are Surface Windspeeds In Knots.
Based On Latitude 35°
225 200 175 150 125 100 75 50 25 0 25 50 75 100 125 150 175 200 225
U_R
U_o Profile Along X'-X'
U_x Profile Along X'-X' (Component of U_o In Direction X)
U_o and U_x Knots
$-X_2$
$+X_1$
Distance From Center Of Storm Nautical Miles

FIGURE 6. HURRICANE HAZEL, OCT. 1954
(Theoretical Surface Wind Velocity Distribution For Stationary Storm)

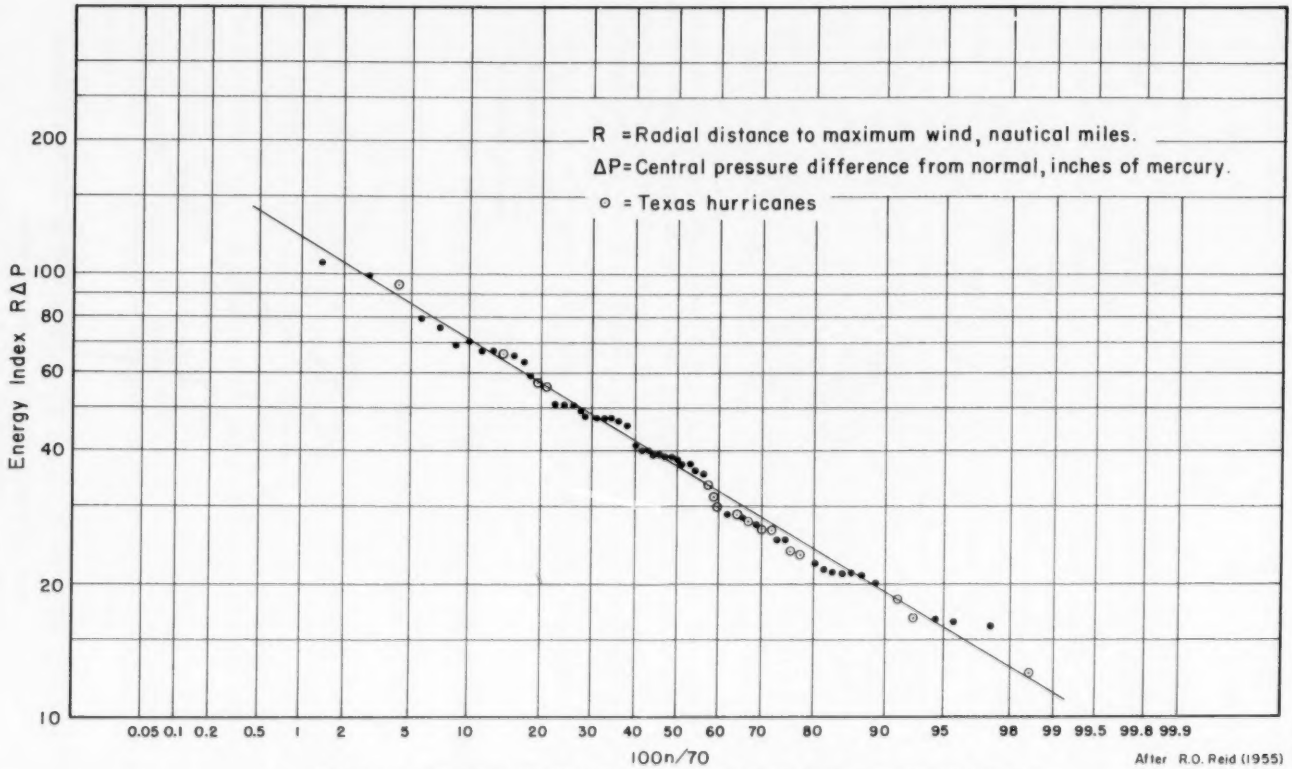

R =Radial distance to maximum wind, nautical miles.
ΔP=Central pressure difference from normal, inches of mercury.
o = Texas hurricanes
Energy Index RΔP
200
100
80
60
40
20
10
0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.5 99.8 99.9
100n/70
After R.O. Reid (1955)

FIGURE 7. ENERGY INDEX RΔP VS. FREQUENCY OF OCCURRENCE
(Based on 69 U.S. Hurricanes, 1900-1949)

In order to establish a relationship between $\int_{x_1}^{x_2} U_x^2 dx$ and $R\Delta p$, it was necessary to analyze a sufficient number of hurricanes covering the range of $R\Delta p$ shown in Figure 7. This was done for 13 U.S. East Coast hurricanes (See Table I) furnished the Beach Erosion Board by the U.S. Weather Bureau.

TABLE I

Summary of 13 Selected Hurricanes off East Coast United States

(Analyzed for Latitude 35°)

No.	Date	R Naut. Miles	Δp In. of Mercury	RΔp	$\int_{x_1}^{x_2} U_x^2 dx$ (knots)^2 N. Miles	Δx N. Miles	U_r Knots	U_R Knots	Actual Storm Speed V_F Knots
1	10/15/54	36	2.31	83.2	488,200	274	43.4	78	26
2	9/19/55	50	1.46	73.0	354,010	253	37.3	60	8.5
3	8/12/55	45	1.40	63.0	323,340	236	37.3	59	7
4	12/2/25	54	1.02	55.1	225,920	207	33.4	48	14
5	9/3/13	39	1.16	45.2	238,300	204	34.5	54	16
6	8/17/55	45	0.82	36.9	160,220	175	30.6	45	15
7	9/17/06	37	0.99	36.6	189,460	180	32.7	50	17
8	8/11/40	27	1.19	32.1	181,960	150	34.9	56	14
9	8/28/11	27	1.05	28.4	159,060	142	32.9	52	9
10	8/14/53	30	0.79	23.7	118,940	131	30.6	45	13
11	10/15/47	12	1.38	16.6	136,250	105	35.8	63	17
12	8/1/44	19.5	0.76	14.8	84,710	87	29.7	45	20
13	8/30/52	12.0	0.95	11.4	83,620	80	30.8	51	12

(Values of R and Δp were furnished by Hydrometeorological Section, U. S. Weather Bureau)

Each hurricane summarized in Table I has been analyzed for the wind field pattern based on a stationary storm for the latitude of 35°. The line of computation X' - X' was selected at a radial distance equal to R. For the most severe storm, that of October 15, 1954, it was found that:

$$\int_{x_1}^{x_2} U_x^2 \, dx$$

was a maximum along this line. The limits of X_1 and X_2 were arbitrarily set at a distance where U_x = 15 knots. If the storm were moving, the maximum value of:

$$\int_{x_1}^{x_2} U_x^2 \, dx$$

would occur at a distance greater than R from the center. Figure 8 shows the energy index relationships based on the 13 hurricanes of Table I, and for all storms except the weaker the following useful relationships are presented:

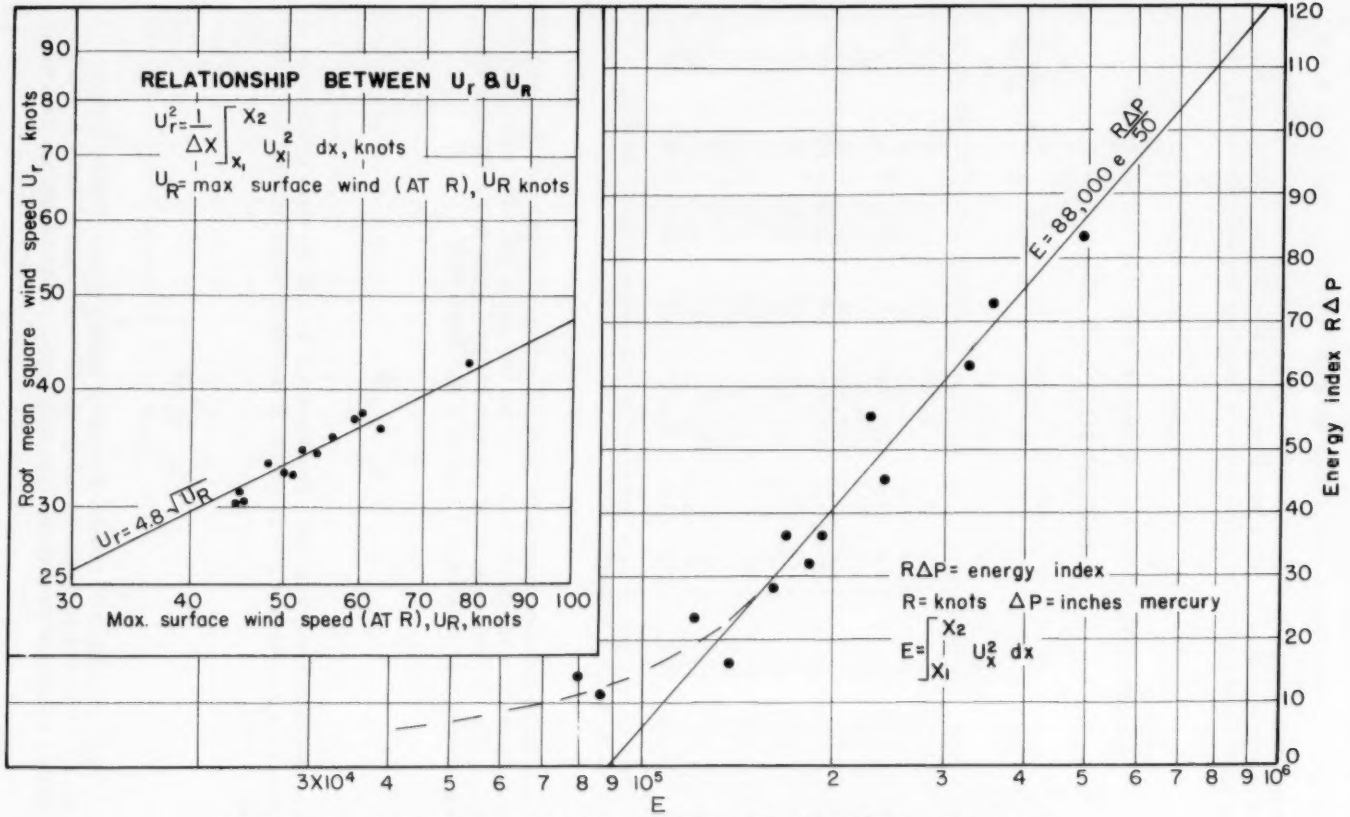

RELATIONSHIP BETWEEN U_r & U_R
$U_r^2 = \frac{1}{\Delta X}\int_{X_1}^{X_2} U_x^2\,dx$, knots
U_R = max surface wind (AT R), U_R knots
Root mean square wind speed U_r knots
Max. surface wind speed (AT R), U_R, knots
$U_r = 4.8\sqrt{U_R}$
$E = 88{,}000\, e^{\frac{R\Delta P}{50}}$
Energy index $R\Delta P$
$R\Delta P$ = energy index
R = knots ΔP = inches mercury
$E = \int_{X_1}^{X_2} U_x^2\,dx$
E

FIGURE 8 HURRICANE ENERGY INDEX RELATIONSHIP $R\Delta P$ vs E

$$10)\ \int_{x_1}^{x_2} U_x^2\, dx = 88{,}000e^{\frac{R\Delta p}{50}}$$

$$11)\ U_r^2 = 23\, U_R$$

} Latitude 35°

where

U_R = sustained wind speed at R, and

$$12)\ U_r^2 = \frac{1}{\Delta X}\int_{x_1}^{x_2} U_x^2\, dx,$$ root mean square wind speed

Figures 12 and 13 are based on latitude 35°. Based on the analysis of 5 of the 13 storms for latitude 25°, it was found that the right side of Equations 10 and 11 should be multiplied by a factor 1.1, which also happens to be the ratio of cos 25° to cos 35°. If the above equations and all following equations are applied at latitude 25° the following correction is required:

multiply H by $\sqrt{1.1}$, T by $\sqrt[4]{1.1}$

Four cases for hurricane wave generation are discussed below.

a. Stationary Hurricane

Using Equation 10 with 1 and 2, one obtains for a stationary hurricane:

$$13)\ H_o = 16.5e^{\frac{R\Delta p}{100}}$$

$$14)\ T_s = 8.6e^{\frac{R\Delta p}{200}},$$ where

} $\phi = 35°$

H_O = the maximum possible value of the significant wave height and T_S = the maximum possible value of the significant wave period for a stationary hurricane. This will occur at a distance from the center equal to the radius of maximum wind.

b. Slowly Moving Hurricane

If one replaced u by U_r in Equation 1, where U_r = root mean square wind speed and superimposes a low steady wind equal to V_F (which could be the forward speed of the hurricane) one obtains:

$$15)\quad H_o = 0.0555\sqrt{U_r{}^2{}_F}\left(1+\frac{V_F}{U_r}\right)$$

$$16)\quad T_s = 0.5\sqrt[4]{U_r{}^2{}_F}\left(1+\frac{V_F}{U_r}\right)^{1/2}$$

$\phi = 35^\circ$

For V_F small compared to U_r, 16 is approximately:

$$17)\quad T_s = 0.5\sqrt[4]{U_r{}^2{}_F}\left(1+\frac{V_F}{2U_r}\right)$$

Equations 15 and 16 are exact for a wind field of constant speed and direction. Now if one assumes V_F is equal to the forward speed of the storm and αV_F is the effective increase in wind speed, then one can make use of relations 10 and 11 with 15 and 17 and obtain:

$$18)\quad H_o = 16.5e^{\frac{R\Delta p}{100}}\left(1+\frac{.208\alpha V_F}{\sqrt{U_R}}\right)$$

$$19)\quad T_s = 8.6e^{\frac{R\Delta p}{200}}\left(1+\frac{.104\alpha V_F}{\sqrt{U_R}}\right)$$

$\phi = 35^\circ$

For a slowly moving storm α is approximately equal to unity, and Equations 18 and 19 will give the maximum values of the significant wave height and period for a slowly moving hurricane. Using $\alpha = 1.0$ tends to compensate for a slight increase in minimum fetch length for a moving storm.

c. Hurricane Moving Forward at the Critical Speed

There exists a critical forward speed of any hurricane which will result in maximum wave generation. If the critical forward speed is exceeded, waves will be lesser in magnitude and period. The critical forward speed is obtained by equating $V_F = 1.52\ T_s$ the group velocity of the significant waves in knots. Substituting $T_{cr} = V_F/1.52 = V_{cr}/1.52$ in Equation 19 one obtains approximately for the critical forward speed:

$$20)\quad V_{cr} = 13.1e^{\frac{R\Delta p}{200}}\left[1-\frac{1.36\alpha e^{\frac{R\Delta p}{200}}}{\sqrt{U_R}}\right]^{-1}$$

$\phi = 35^\circ$

Using $V_F = V_{cr}$ from 20, equations 18 and 19 will give the critical values of the significant wave heights and period respectively, that is the maximum possible values of H_o and T_s. The above equations have been used to obtain wave heights and periods for the 13 storms given in Table I, where α was assumed equal to 1.0. The results are summarized in Table II and Figures 9 and 10 respectively. Actual storm speeds are given in Table I. From Figures 9 and 10 one can read off the corresponding heights and periods. These results are given below as H_{ac} and T_{ac} in Table II. In general, actual heights and periods are less than the maximum possible values as given for critical forward speed.

TABLE II

Summary of Highest Deep Water Significant Waves Predicted for 13 Selected Hurricanes off East Coast United States

		Stationary Storm		Storm Moving at Actual Speed (See Table I)		Storm Moving at Critical Speed				
		H_o	T_s	H_{ac}	T_{ac}	V_{cr}	H_{cr}	T_{cr}	$\frac{H_{cr}}{H_o}$(Sta)	$\frac{V_{cr}}{T_s}$(Sta)
No.	Date	Feet	Sec.	Feet	Sec.	Knots	Feet	Sec.		
1	10/15/54	37.9	13.0	59	16	24.8	59.8	16.4	1.58	1.91
2	9/19/55	34.2	12.4	42	13.5	24.1	56.4	15.9	1.65	1.94
3	8/12/55	31.0	11.8	37	13.0	22.7	50.0	15.1	1.61	1.92
4	12/2/25	28.7	11.3	40.5	13.5	22.0	47.7	14.6	1.66	1.95
5	9/3/13	26.0	10.8	37.5	13.0	20.7	42.1	13.7	1.62	1.92
6	8/17/55	23.9	10.3	35.0	12.5	20.0	39.0	13.2	1.63	1.94
7	9/17/06	23.5	10.2	35.5	12.5	19.3	37.2	12.8	1.58	1.89
8	8/11/40	22.6	10.1	31.5	12.0	18.7	34.7	12.4	1.54	1.85
9	8/28/11	22.4	9.9	28.0	11.0	18.6	34.7	12.3	1.55	1.88
10	8/14/53	20.8	9.7	29.0	11.5	18.5	29.5	12.2	1.42	1.91
11	10/15/47	19.5	9.4	28.0	11.0	17.2	28.2	11.4	1.45	1.83
12	8/1/44	19.1	9.3	28.0	11.0	17.5	29.5	11.5	1.55	1.88
13	8/30/52	18.5	9.2	25.0	10.5	17.1	27.6	11.3	1.49	1.86
									1.56	1.90

From the data in Table II, the following deductions are made:

a) The critical forward speed of the storm in knots is equal to about 1.9 times the value of the significant period for the stationary storm.

b) The critical significant wave height is equal to about 1.56 times the value of the significant wave height for the stationary storm.

c) The critical significant wave period is equal to about 1.25 times the value of the significant wave period for the stationary storm.

From the above three statements one finds:

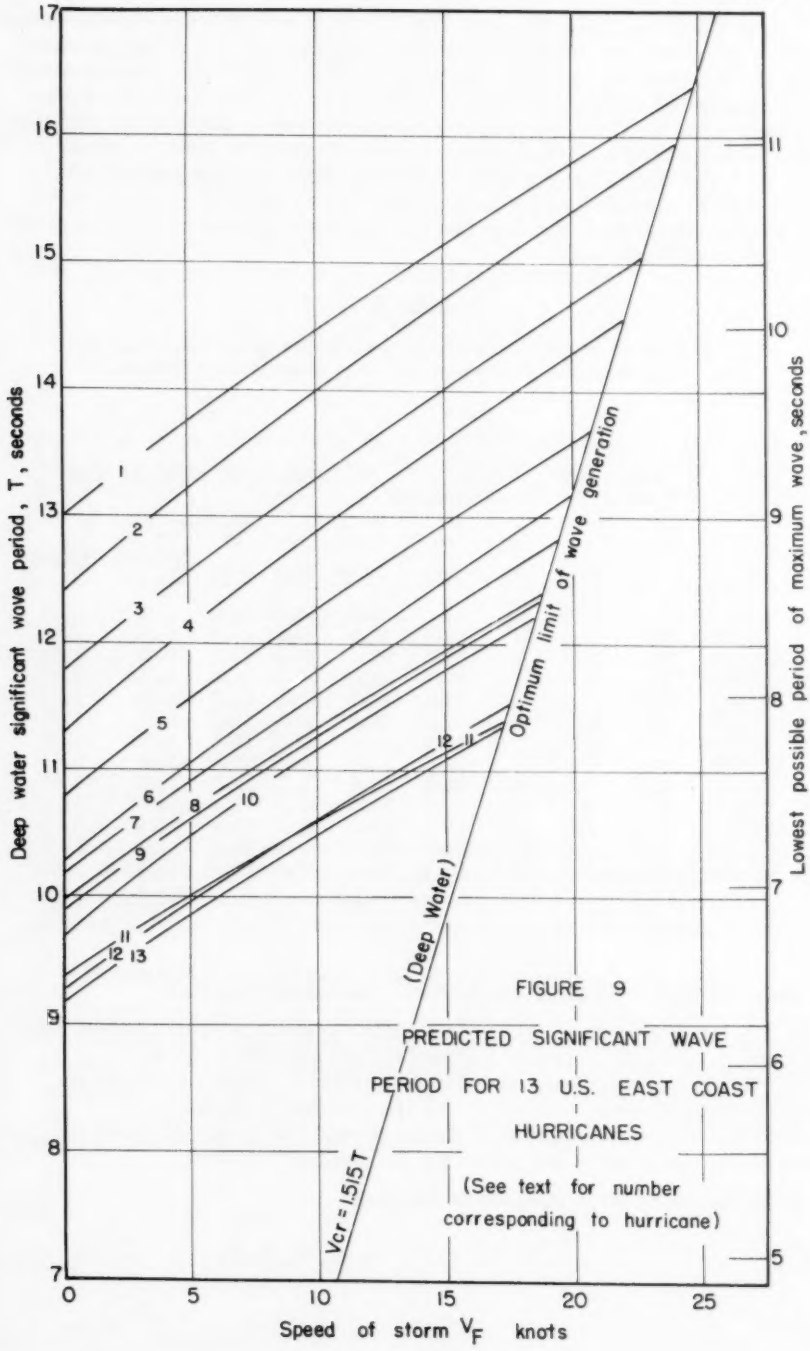


FIGURE 9

PREDICTED SIGNIFICANT WAVE PERIOD FOR 13 U.S. EAST COAST HURRICANES

(See text for number corresponding to hurricane)

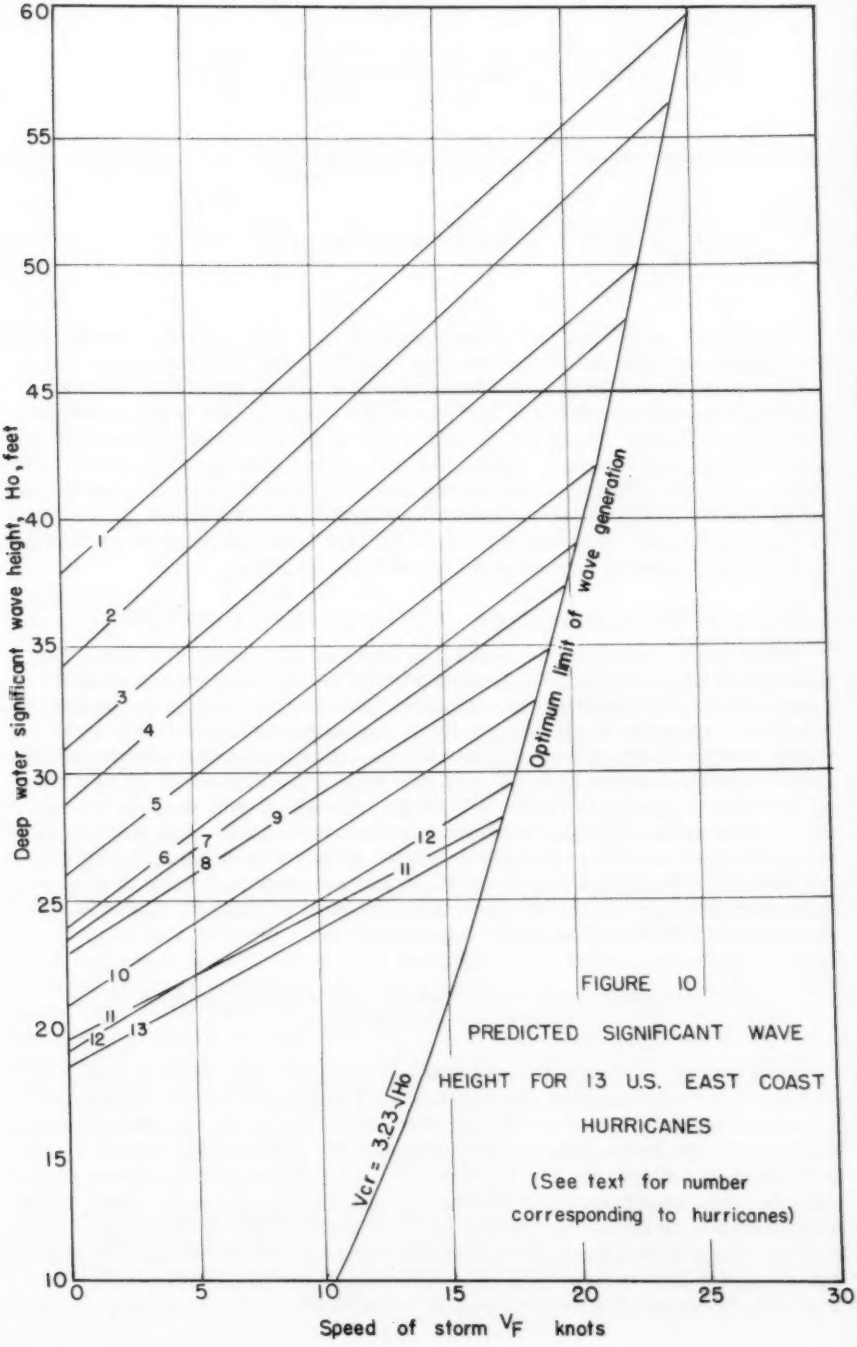


FIGURE 10

PREDICTED SIGNIFICANT WAVE HEIGHT FOR 13 U.S. EAST COAST HURRICANES

(See text for number corresponding to hurricanes)

21) $V_{cr} = 16.3e^{\frac{R\Delta p}{200}}$, critical forward speed

22) $H_{cr} = 25.8e^{\frac{R\Delta p}{100}}$, critical significant wave height

23) $T_{cr} = 10.7e^{\frac{R\Delta p}{200}}$, critical significant wave period

} $\phi = 35^{\circ}$

It might be mentioned that if one assumed $\alpha = 0.75$ equations above would be multiplied by the factor 0.95 for V_{cr} and T_{cr} and 0.91 for H_{cr}.

Equations 22 and 23 give maximum possible values for significant wave height and period respectively that can be generated in any hurricane passing critical conditions. These waves will occur in the right rear quadrant of the hurricane moving forward. Conditions for critical wave generation are probably never fully attained, since the duration required at critical speed is generally insufficient prior to the storm moving forward at a speed greater than critical speed, and α is less than 1.0. Another factor to consider is that the storm may change its course prior to critical conditions.

d. Hurricane Moving at a Forward Speed Greater Than Critical Speed

Under these conditions the preceding development cannot continue. The assumption of $\alpha = 1.0$ can no longer be used to represent an added wind speed effect. Furthermore, the storm will be moving ahead of the maximum waves now acting as swell but still under the influence of relatively high winds. Only a rough rule of the thumb is used, and that is the highest significant waves for a storm speed $V_{cr} + \Delta V$ will be of same order of magnitude as those for $V_{cr} - \Delta V$, where ΔV is the increase in storm speed beyond the critical value. In order to obtain a better answer one might make use of the space-time wind field technique and the graphical method proposed by WILSON.(10) Even here one would either have to assume a value of α, or analyze in great detail all available synoptic weather maps for a storm with similar characteristics as the design storm.

Maximum Deep Water Waves in the Pacific Typhoon of September 26, 1935

In regards to verification of the above techniques, little data is available on deep water wave heights in a hurricane. Perhaps the best available data at present for this type of storm are those observed by the Imperial Japanese Navy during the Pacific typhoon of September 26, 1935, a summary of which is given by H. ORAKAWA and K. SUDA.(12) The center of this typhoon passed over the main squadron, about 15 nautical miles to the left of it. Other ships encountered various sections of the typhoon, and a complete coverage of wind and wave data were taken. Nine different ships reported observations of highest waves encountered, and a total of 13 observations appear in publication.(11) These are reproduced in part in Table III.

TABLE III

Summary of Highest Waves Observed by the Imperial Japanese Navy
In the Typhoon off East Japan September 26, 1935
(From Monthly Weather Review)

Name of Ship	Time 135th Meridian Civil Time	Observed Highest Waves H Feet	L Feet	C Feet/Sec.	T Sec.	Computed Period T=L/C	$T=\sqrt{\frac{L}{5.12}}$
Hosho	1215	33					
Mikuma	1250	26	590	47	13	12.5	10.70
Hosho	1400	46					
Hosho	1420	58					
Hoguro	1418	49	1150(about)				15.0
Mikuma	1445	33	658	47	13.3	14.0	11.4
Amogini	1458	82	985				
Asakaze	1500	49	650-1000(estimated)				
Nachi	1500	42-46	395	29.5	9.0	13.4	8.8
Tsorurd	1500	33	658				11.3
Naka	1522	46					
H_{ave} = 45.4 all points H_{ave} = 43.3 (all points, except highese and lowest)							
		Pyramid	Waves				
Susaki	1540	65-98	490				9.8
Nachi	1550	42-46	395	26	9	15.2	8.8

For this particular storm the radius of maximum wind is equal to about 50 nautical miles and Δp about 1.63 inches of mercury. Using Figure 3 one obtains U_R = 65 knots. During the period of observation the forward speed of the storm was equal to or greater than 39 knots, and the maximum sustained wind speed at the radial distance R was greater than 40 meters per second (86 knots) but less than 45 meters per second (97 knots). Using the above values of R, Δp, and U_R maximum values of the significant waves were predicted. The results are compared with the observations in Figure 11. The critical storm speed for maximum waves computed from Equation 20 is about 25 knots whereas the forward speed of the storm during observations was greater than 40 knots. For this reason it is expected the observations to be less than the critical height. An estimate of the heights and periods are of the same order of magnitude as those given for $V_F = V_{cr} - (40 - V_{cr}) = 10$ knots. This would indicate excellent agreement between observation and forecasts, provided one considers the average of all observations which is equal to 45.4 feet as indicative of the significant wave. The maximum wave observed is about 82 feet, from which:

$$\frac{H_{max}}{H_s} = \frac{82}{45.4} = 18 \quad \text{which is also reasonable.}$$

The lowest possible period for maximum wave is $T^1 = \sqrt{\frac{82}{.877}} = 9.7$ seconds. This would indicate the wave to be near breaking and from $H/L = 1/7$ one finds the lowest possible wave length to be equal to $L = 82 \times 7 = 574$ feet. The above values of $T = 9.7$ seconds and $L = 574$ feet are in fairly good

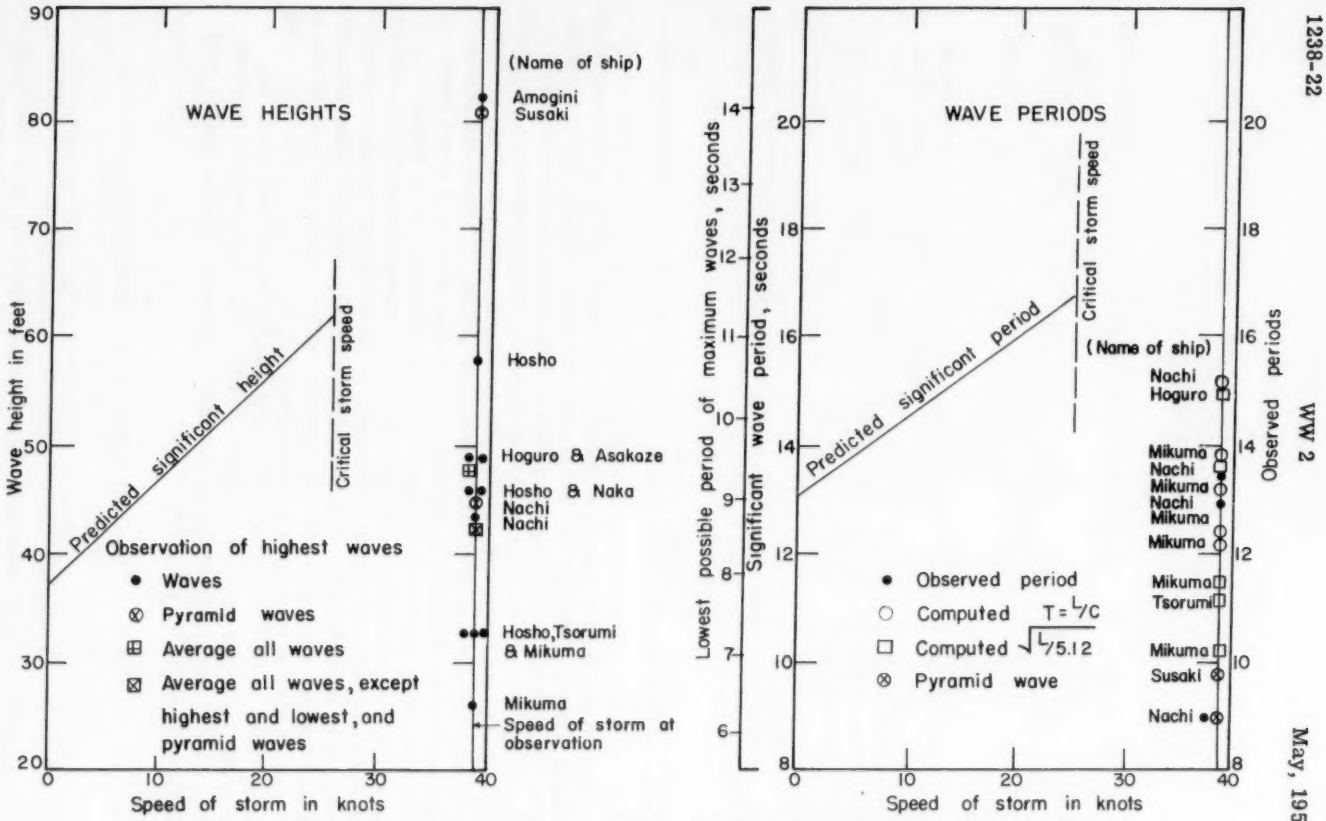


FIGURE 11 PREDICTED AND OBSERVED WAVE FOR PACIFIC TYPHOON SEPT. 26, 1935

agreement for the maximum wave given in Table IV, considering difficulties involved in observing waves from aboard ship.

Unfortunately actual recorded wave data within a hurricane or typhoon are non-existent at present, and the preceding verification, although the best available, must still be considered somewhat tentative.

Generation of Wind Waves in Shallow Water

The foregoing procedures are used for deep water, whereas actual design conditions are more frequent for shallow water. However, it is first necessary to predict deep water waves, which are then propagated across the continental shelf taking bottom friction and refraction into account. Hence, the accuracy of wave forecasts in shallow water can be no more accurate than the deep water wave forecast.

Wave forecasting techniques for shallow water are not so numerous as for deep water. Some of the earlier work was by THIJSEE and SCHIJF[13] wherein are presented dimensionless relationships for obtaining wave heights and velocities as functions of relative depth as well as relative fetch length. A similar set of relationships were obtained by BRETSCHNEIDER[3] using numerical means of combining the relationships of wave generation[5] with the relationships for wave energy dissipation due to bottom friction proposed by BRETSCHNEIDER and REID.[14] Observations obtained in the shallow water of the Gulf of Mexico[15] and also from Lake Okeechobee, Florida[16] show fairly good agreement with this technique, provided a bottom friction factor of .01 is selected. Such a factor as the friction factor entails other considerations, since the theory is based on the Airy wave theory, which cannot be considered representative of high waves in a complex sea under high wind conditions. Probably it is better to state that the bottom friction factor of .01 is actually an apparent or calibration friction factor, which when used with the equation for shallow water wave computations will result in verification with observed data.

This being the case for known conditions[15, 16] techniques for forecasting waves in shallow water are extended to the continental shelf. One can expect fairly reasonable results, provided the slope of the continental shelf is not too great; flatter than 5 to 10 feet per mile.

It is not intended to go into detail material presented in reference 3, 14, and 15. Instead an actual example is worked out in detail for obtaining a hurricane design wave.

Example Forecast of Design Wave (Based on Hurricane Hazel of October 15, 1954)

As a hurricane moves over the continental shelf, two cases arise, both of which must be considered. First, the initial deep water waves generated may be propagated shoreward as swell under the continued influence of hurricane winds; and second, regeneration of wind waves is constantly taking place as the hurricane moves over the continental shelf. Swell will feel bottom far from shore and commence losing energy at an early stage, whereas wind waves with shorter period continue to grow and do not feel bottom until they are sufficiently large and are nearing the coast. It is therefore necessary to determine whether the swell or the wind wave is more critical. Refraction may determine the final design wave.

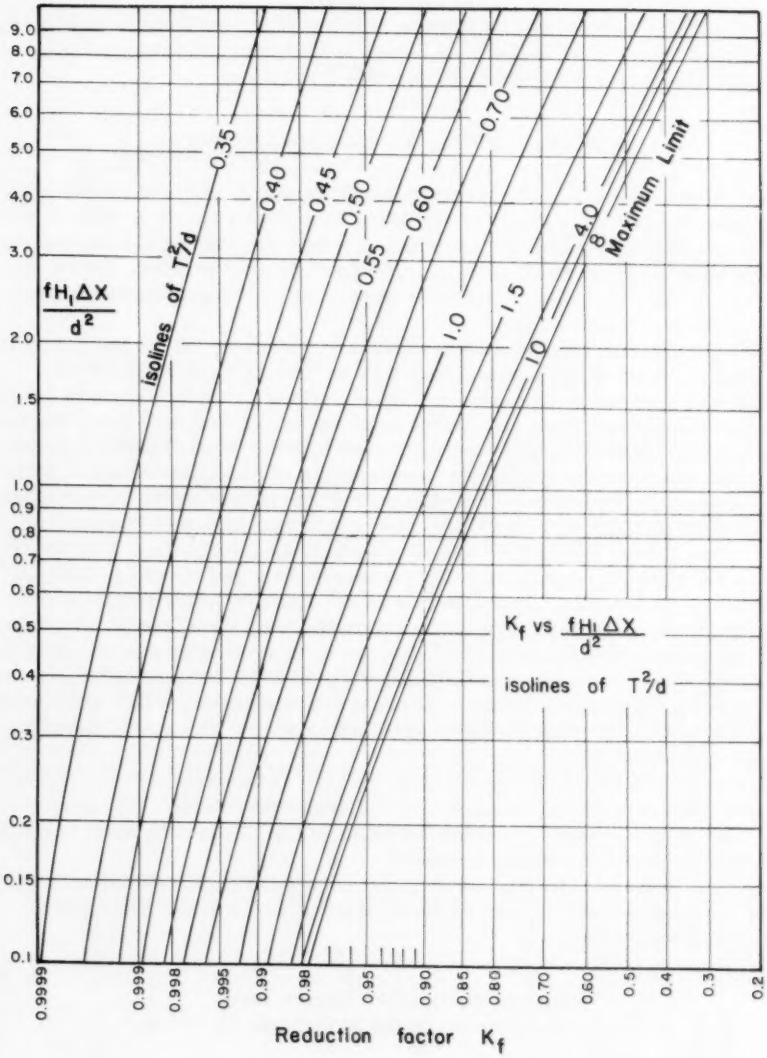


FIGURE 12 RELATIONSHIP FOR FRICTION LOSS OVER A BOTTOM OF CONSTANT DEPTH

Once the design hurricane is selected and maximum deep water waves computed, it is necessary to select a path across the continental shelf, over which shallow water computations are made. A path perpendicular to the coast is usually critical. Storm tide computations (wind set-up and pressure surge) are made, and refraction diagrams constructed.

Shallow water computations consist of the following steps:

(a) Maximum wind over shallow water is selected equal to $U_R + V_F$.

(b) Equivalent deep water fetch length F_e, is obtained from Equation 1:

$$24) \quad F_e = \left[\frac{H_o}{.0555\ (U_R + V_F)} \right]^2$$

This in effect represents that fetch required under steady wind conditions which (the actual minimum fetch in shallow water must always be equal to or less than F_e) gives the same wave energy as Equation 18.

(c) The traverse is segmented into equal increments, ΔF, usually about 5-10 miles in length, depending on the bottom slope (see Figure 13).

(d) An average depth, d_{ave}, is determined over each increment.

(e) A deep water wave height, H_o is computed at the beginning of the first increment of ΔF.

(f) This value of H_o is then assumed to travel over the increment ΔF as swell, taking bottom friction into account. This is done by use of Figure 12, from which a reduction factor K_f is determined. The actual significant wave height at the end of the increment becomes equal to $H_s = H_o K_f K_s K_r$ where K_s is the shoaling factor and K_r is the amount of refraction taking place over the increment ΔF.

(g) An equivalent deep water wave height H'_o is obtained from $H'_o = K_f H_o$.

(h) An equivalent deep water fetch length F'_e is obtained from:

$$25) \quad F'_e = \left[\frac{H'_o}{.0555\ (U_R + V_F)} \right]^2$$

(i) An equivalent deep water wave height is computed at the end of the second increment using $F = F'_e + \Delta F \leq F_e$, using Equation 1. An average wave height over this increment is computed using H'_{o1} and H'_{o2}, where H'_{o1} is the equivalent deep water height at the beginning of the increment and H'_{o2} at the end of the increment.

(j) With the average wave height $1/2\ (H'_{o1} + H'_{o2})$ steps f, g, h, and i are repeated. This is done for all except that last increment or until the wave breaks whichever occurs first. Usually the last increment cannot be treated by the above method since here the bottom slope increases too rapidly shoreward.

(k) The maximum wave height for all except the last increment is computed from:

$$26) \quad H_{max} = H_s \left[\frac{1.45gd}{U^2} \right]^{.1} \quad \pm 10\% \leq .78d \leq 1.87H_s$$

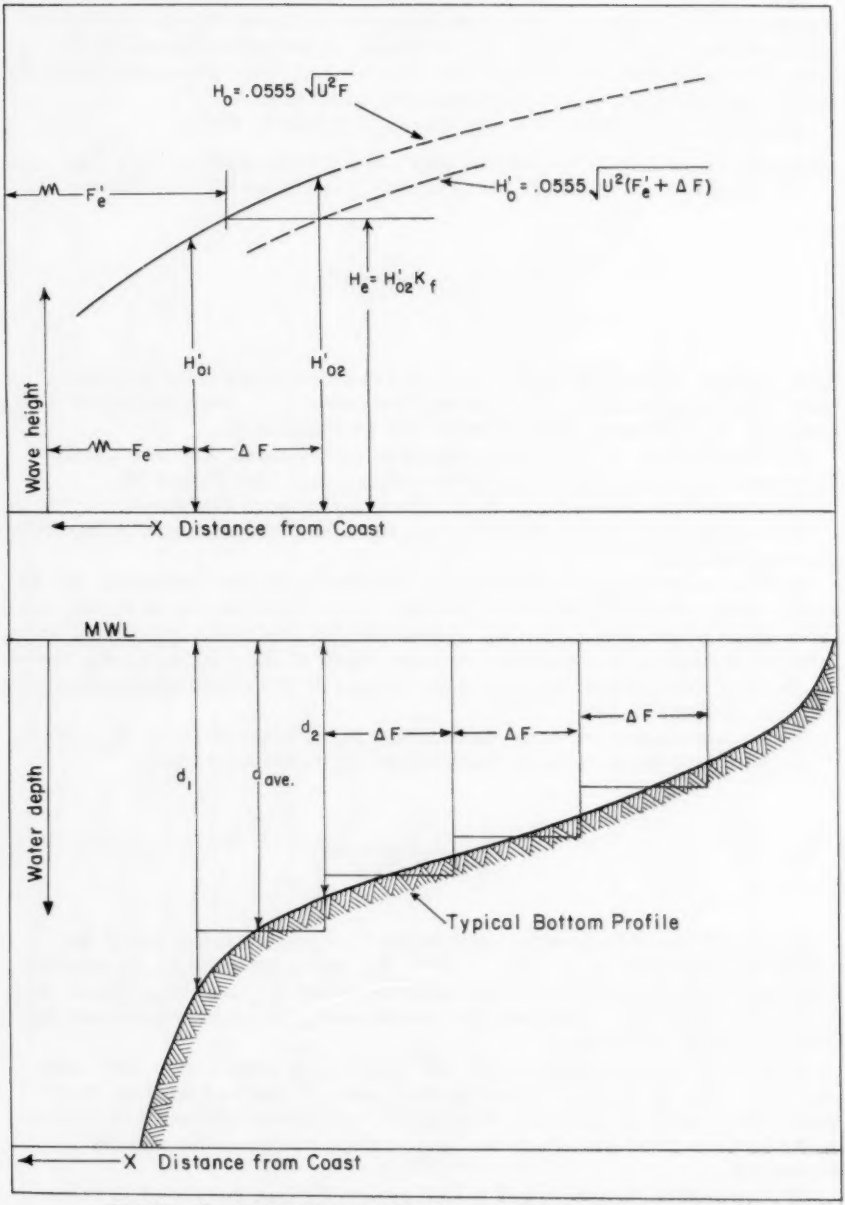


FIGURE 13 SCHEMATIC DIAGRAMS ILLUSTRATING PROCEDURE FOR COMPUTING WIND WAVES IN SHALLOW WATER

The above formula is based on wind generated waves recorded in the Gulf of Mexico[15] and hurricane wind generated waves recorded in Lake Okeechobee, Florida.[16]

(l) Maximum waves over the last increment and the location of breaking is obtained by plotting $H_b = .78d$ seaward from the coast. A continuation of the smooth curve of H_{max} obtained in (k) is made until it intersects the curve of $H_b = .78d$.

Results of computations for maximum swell for hurricane Hazel are presented in Figure 13 and Table V. Storm tide computations were made in accordance with Reference 1. Because of the work entailed refraction was not considered in this example, and the actual wave height may be higher or lower than computed, depending on the amount of refraction.

A similar process is used in obtaining the maximum wind wave being regenerated over the continental shelf. In this case one does not use a constant period T_s, but a new period over each increment is obtained by using Equation 2 in a similar manner as Equation 1. The results of these computations are presented in Table VI. It is noted from Table V and VI that the swell height and wind wave height are almost the same, which indicates that wave energy loss for shallow water wind waves is practically independent of wave period. This is only true where the bottom slope does not break off and becomes very steep at relatively shallow depths.

SUMMARY AND CONCLUSIONS

This paper, representing the latest techniques proposed for obtaining the Hurricane Design Wave, is intended to supplement the information presented in Reference 1. The material is based on experience gained using Reference 1, and is also based on additional research and supplemental data.

The concept of "Energy Index" is undoubtedly a great contribution for classifying hurricanes. The history of a hurricane is most confusing. However, once a hurricane is formed its characteristics are very similar to its predecessors or its followers. In fact, for any particular latitude a hurricane may be characterized by the radius of maximum wind, R, the pressure anomally at the center from normal pressure, Δp, and the forward speed, V_F. The kinetic energy of a hurricane is proportional to $R\Delta p$, and therefore can be used as an index of energy. Wave energy on the other hand is proportional to H^2 and from Equation 1 one sees that H^2 is proportional to U^2F. This lead to the Energy Index relationships, $H^2 \sim U^2F = f(R\Delta p)$. In the final development for maximum deep water waves, it is found that any hurricane has a critical speed which if reached will generate the largest waves, and if exceeded will result in waves smaller than those under critical conditions.

The technique used for computing hurricane waves over the continental shelf taking bottom friction into account represents an extension of a method used for computing hurricane wind waves in shallow water of constant depth. This method has been "calibrated" using hurricane wave data recorded in Lake Okeechobee, Florida, and it is believed to be the best available to date. However, it is expected that improvement here can be made as more information on wind and waves are obtained within a hurricane, both for deep and shallow water.

TABLE IV - SUMMARY OF COMPUTATIONS FOR HURRICANE SWELL IN SHALLOW, UNDER INFLUENCE OF WIND FOR DESIGN STORM

X n. miles	63	55	50	45	40	35	30	25	20	15	10
$(X + \Delta X)$	55	50	45	40	35	30	25	20	15	10	5
MLWL at X	600	150	137	125	115	107	98	90	82	74	61
MLWL at $(X+\Delta X)$	150	137	125	115	107	98	90	82	74	61	47
S (wind set-up)	0.3	0.8	1.3	1.8	2.4	3.1	3.8	4.6	5.4	6.3	7.5
F (n. mile)	110	108.7	91.0	78.58	66.57	61.82	50.12	40.98	33.43	27.00	21.09
d_{t1} (feet)	605.3	155.8	143.3	131.8	122.4	115.1	106.8	99.6	92.4	85.3	73.5
d_{t2} (feet)	155.8	143.3	131.8	122.4	115.1	106.8	99.6	92.4	85.3	73.5	59.7
$d_{t\ ave}$ (feet)	380.5	149.4	136.6	127.0	117.7	109.0	103.2	96.0	88.9	79.4	66.6
H_{o1} (feet)	56.6	56.3	51.5	47.9	46.9	42.4	38.2	34.61	31.2	28.1	24.8
H_{o2} (feet)	56.6	56.6	52.9	49.5	48.3	44.2	40.2	36.6	33.4	30.5	27.6
$H_{o\ ave}$ (feet)	56.6	56.15	52.2	48.7	47.6	43.3	39.2	35.6	32.3	29.3	26.2
$T^2/d_{t\ ave}$	.673	1.714	1.874	2.016	2.175	2.349	2.480	2.667	2.880	3.224	3.844
$K_{s\ ave}$	.974	.928	.933	.938	.945	.952	.958	.967	.975	.993	1.020
$A=fH_{o\ ave}\Delta X K_s/d^2_{ave}$	.1801	.7097	.7935	.8610	.9871	1.055	1.072	1.136	1.211	1.460	1.832
K_f	.994	.910	.905	.890	.875	.865	.860	.853	.840	.813	.782
H'_{o2} (feet)	56.3	51.51	47.87	44.06	42.26	38.23	34.57	31.22	28.06	24.80	21.58
F_e (n. miles)	108.7	91.0	78.58	66.57	61.82	50.12	40.98	33.43	27.00	21.09	15.97
K_{s2}	.923	.929	.935	.940	.949	.955	.962	.972	.981	1.004	1.041
H_s (at $X+\Delta X$)	51.96	47.85	44.76	41.42	40.10	36.51	33.26	30.35	27.53	24.90	22.47
H_{max}/H_s	1.53	1.515	1.50	1.49	1.480	1.47	1.46	1.450	1.44	1.42	1.40
H_{max} (at $X+\Delta X$)	79.4	72.3	67.1	61.6	58.2	53.6	48.6	44.2	39.6	35.4	31.50

TABLE V - SUMMARY OF COMPUTATIONS FOR HURRICANE WIND WAVES IN SHALLOW WATER, DESIGN STORM
(Continuation of Table V)

X n. miles	63	55	50	45	40	35	30	25	20	15	10
H_{o1} (feet)	56.6	56.3	51.9	47.6	44.0	40.6	38.0	34.5	32.6	29.8	27.1
H_{o2} (feet)	56.6	56.6	53.3	49.2	45.6	42.4	39.4	36.9	34.8	32.2	29.7
$H_{o\ ave}$ (feet)	56.6	56.45	52.6	48.4	44.8	41.5	38.7	35.7	33.7	31.0	28.4
T_{o1} (sec)	16.00	16.0	15.34	14.70	14.13	13.56	13.15	12.50	12.15	11.63	11.09
T_{o2} (sec)	16.00	16.0	15.55	14.94	14.39	39.96	13.37	12.96	12.57	12.09	11.61
$T_{o\ ave}$ (sec)	16.00	16.0	15.45	14.82	14.26	13.76	13.26	12.73	12.36	11.86	11.35
$T_{o\ ave}^2 / d_{t\ ave}$	.673	1.714	1.735	1.728	1.728	1.721	1.704	1.688	1.718	1.772	1.934
$K_{s\ ave}$	.947	.926	.928	.930	.929	.924	.929	.924	.926	.923	.932
$A=fH_{o\ ave}\Delta X K_s / d^2_{ave}$	.1801	.712	.784	.861	.913	.963	1.026	1.088	1.240	1.380	1.814
K_f	.994	.917	.893	.895	.890	.885	.885	.885	.858	.840	.795
H'_{o2} (feet)	56.3	51.90	47.60	44.03	40.58	37.52	34.87	32.66	29.86	27.05	23.61
F_e (n. mile)	108.7	92.37	77.70	66.48	56.47	48.28	41.70	36.58	30.58	25.09	19.12
T'_{o2} (sec)	16.0	15.31	14.7	14.13	13.56	13.15	12.50	12.15	11.63	11.09	10.35
K_{s2}	.949	.926	.925	.923	.921	.920	.920	.920	.920	.923	.929
$H_{1/3}$ (at X+ΔX)	53.43	48.06	44.03	40.64	37.37	34.52	32.08	30.05	27.47	24.97	21.93
$H_{max}/H_{1/3}$	1.53	1.515	1.50	1.49	1.48	1.47	1.46	1.45	1.44	1.42	1.40
H_{max} (at X+ΔX)	81.6	73	66	60.5	55.2	50.8	46.8	43.6	39.6	35.4	30.7

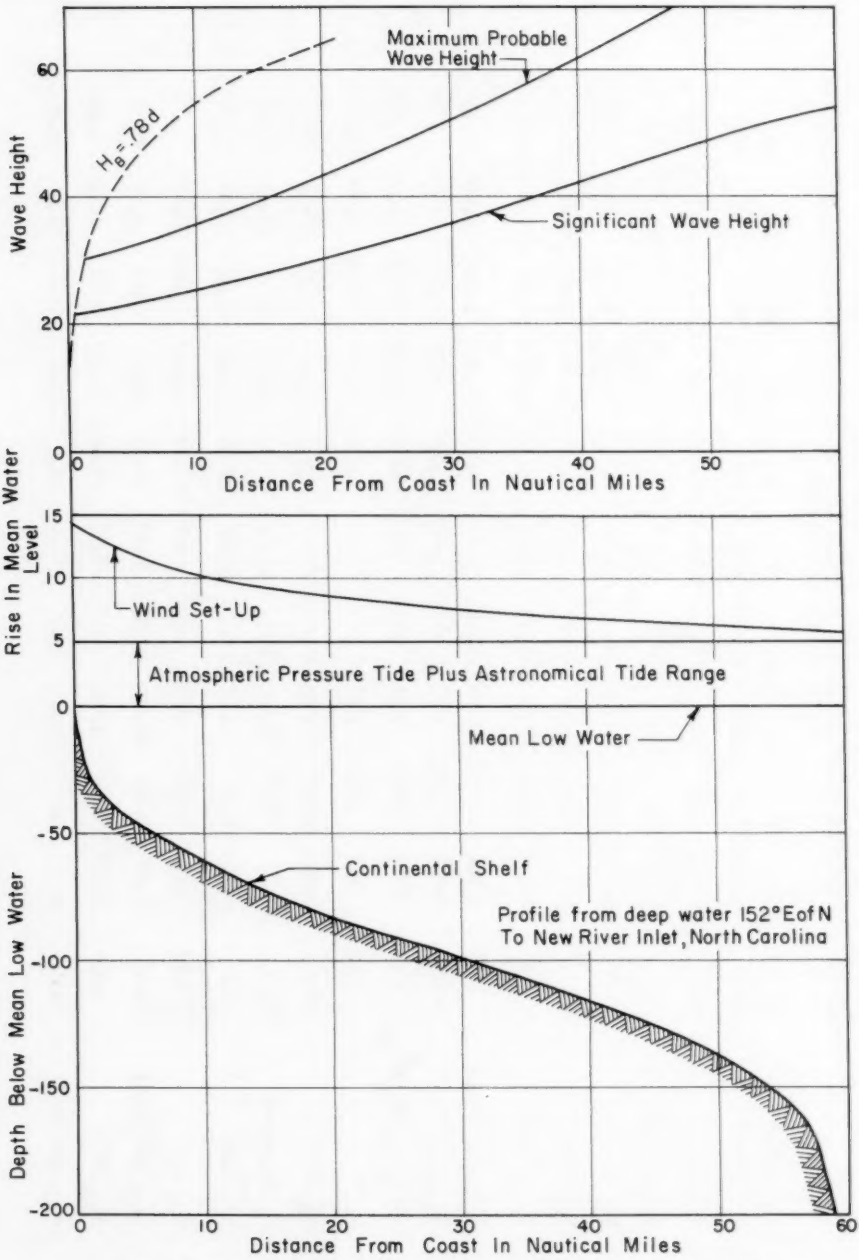


FIGURE 14. HURRICANE WAVE FORECAST FOR STORM MOVING PERPENDICULAR TO COAST

(Based on wind and pressure field of Hurricane Hazel, Oct. 15, 1954)

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LIST OF SYMBOLS

C	Wave speed, feet/second, also in knots
C_g	Group velocity, knots
d	Depth of water, feet
dx	Derivative of x
e	Base of natural logarithm = 2.7183
F	Fetch length, nautical miles
F_{min}	Minimum fetch length, nautical miles
F_e	Equivalent deep water fetch length in nautical miles for generation of hurricane deep water waves
F'_e	Equivalent deep water fetch length in nautical miles for generation of equivalent deep water wave height, H'_0, in shallow water
f	Bottom friction factor, also Coriolis parameter
g	Acceleration of gravity, 32.16 feet/sec.^{2}
H	Wave height, feet
H_0	Deep water significant wave height, feet
H'_0	Equivalent deep water significant wave height, feet
H_s	Significant wave height in shallow water, feet
H_{cr}	Critical value of significant wave height, feet
H_{max}	Maximum probable wave height, feet
K_s	Shoaling factor
K_r	Refraction coefficient
K_f	Coefficient for reduction in wave height for wave energy loss due to bottom friction
L	Wave length, feet
p	Atmospheric pressure, inches of mercury
r	Radial distance (in storm) to occurrence of gradient and/or surface wind, nautical miles

R	Radius of maximum wind, nautical miles
S	Storm tide, feet
T	Wave period, seconds
T_s	Significant wave period, seconds
T'	Lowest possible period of maximum wave, seconds
T_{cr}	Critical value of significant wave period, seconds
t	Duration of wind, seconds or hours
t_{min}	Minimum duration of wind, hours
U	Horizontal surface wind speed (constant), feet/second and knots
U_x	Surface wind speed component in direction X, knots
U_r	Root mean square wind speed, over a fetch of variable wind speed, knots
U_R	Maximum sustained wind speed (at distance from center equal to R) for a stationary hurricane, knots
V_g	Gradient wind speed, knots
V_c	Cyclostrophic wind speed, knots
V_F	Forward speed of hurricane, knots
V_{cr}	Critical forward speed of hurricane, knots
α	Factor equal to or less than unity to be multiplied by V_F to obtain increase in wind speed due to forward speed of hurricane
ΔX	Increment of horizontal distance, feet and nautical miles
ΔF	Increment of fetch length, nautical miles
Δp	Pressure anomally at center of hurricane from normal pressure, inches of mercury
ρ	Density of air

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MODERN FACILITIES FOR OHIO RIVER NAVIGATION[1]

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(Proc. Paper 1239)

SYNOPSIS

Physical deterioration, obsolescence and increasing demands of traffic make essential the modernization and replacement of existing navigation facilities on the Ohio River and certain major tributaries. General requirements and criteria establishing the characteristics of replacement facilities, now in various stages of construction and planning, are presented in this paper.

INTRODUCTION

The Ohio River is subject to great variation in river stages. For portions of the year, ranging from about 30 percent in the upper river to 50 percent or more in the lower river, there is sufficient flow in the natural river to accommodate craft with a draft of nine feet or more. However, during periods of relatively low flows, navigation in the unimproved river was impaired. Early efforts to enhance the capability of the river for continuing navigation throughout each year were confined to removal of snags, rocks, and similar obstructions, and to the construction of control works, such as diversion and restraining dikes, for the purpose of directing and concentrating available river flows in established narrow channels. However, as the use of the river

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for commerce grew, such methods of improvement proved to be inadequate. Thus, as a result of studies made during the period 1870 to 1875, the construction of a system of locks and dams was initiated to insure adequate navigable depths throughout each year. Continuing studies and subsequent modifications and extensions of the planned system provided the complete lock and dam system in use on the Ohio River in 1956. The system assures a navigable depth of nine feet throughout the length of the river from Pittsburgh, Pennsylvania, to its mouth near Cairo, Illinois.

The existing navigation system comprises forty-six locks and dams. Forty-one of these structures are of the same general type. Each includes one 110 x 600-foot lock chamber and a wicket dam, in which beartrap regulatory works are usually incorporated. The wickets of the dam can be lowered to the bed of the river when flows are sufficient to provide the required nine-foot navigable depth. In such instances tows pass over the dams without the necessity of transit through the locks. The regulatory works are used to control the pool levels at the dams during relatively minor flow fluctuations. The lifts at the dams average about seven feet. Three of the remaining five structures—Emsworth, Montgomery, and Gallipolis Locks and Dams—each provide dual locks and a non-navigable gated-crest dam. Dashields Locks and Dam comprises dual locks and a fixed non-navigable overflow dam. The size of the main locks at the above-named four dams is 110 x 600 feet. The size of the auxiliary chambers is 56 x 360 feet at all of these dams except Gallipolis where a 110 x 360-foot chamber has been provided. Locks and Dam No. 41 at Louisville, Kentucky, includes dual locks and a combined fixed weir and wicket dam with beartrap regulatory works. The size of the main lock chamber is 110 x 600 feet and that of the auxiliary chamber is 56 x 360 feet. The lifts at these latter five dams vary as follows: Emsworth—18 feet; Dashields—10 feet; Montgomery—19.4 feet; Gallipolis—26 feet; and Dam No. 41—37 feet. Except for Montgomery Locks and Dam and Gallipolis Locks and Dam, which were completed in 1936 and 1938, respectively, all of the structures in the existing system were completed in 1929 or earlier.

Early traffic movement on the Ohio was virtually all downbound, destined for lower Ohio and Mississippi River ports. The adoption of the wicket dam and a 110 x 600-foot lock for general use on the Ohio River represents a compromise between the desire of navigation interests for an unimpaired movement of downbound traffic and the necessity for a system improvement to assure the navigable depths which they desired.

It is interesting to note that all of the working parts of the wicket dams and their regulatory works are permanently submerged. Under this condition proper inspection is difficult and maintenance and repairs are expensive. Also, the wickets must be manually operated, resulting in high operation costs when compared to modern structures designed for powered operation.

At the time the existing lock and dam system was conceived, it was considered that a movement of about 13,000,000 tons of commerce annually would justify the cost of the system. However, traffic volume has exceeded all expectations since completion of the system. As the economy of the Ohio River Valley has expanded, upbound traffic movements have become almost equal to downbound movements. Powerful diesel-driven multiple-screw towboats have replaced virtually all of their paddle-wheel ancestors. The average length of tows has increased steadily. Very few long-haul toes are less than 600 feet in length and tows that are 800 to 1000 feet or more in length are prevalent. Thus, while the existing system has performed admirably since its completion,

it cannot be expected to continue to satisfy adequately and efficiently the ever-growing traffic demand.

Necessity for Replacing Existing Facilities

There are several factors which combine to establish the essentiality for replacing facilities existing in 1956 on the main stem of the Ohio. The physical deterioration of structures under increasingly heavy usage, the growing cost of maintaining and operating the large number of structures, the time-consuming operations of numerous passages through locks only about half the size of the most economical tow for large-volume service, and the resistance to propulsion through relatively shallow pools are factors sufficient to warrant initiation of a modernization program. But there is one factor, which would have been regarded as bordering on the fantastic a few years ago, that now gives evidence of becoming the most over-riding necessity of all for replacing and modernizing the existing facilities. This is the physical inability to pass traffic at individual locks at the rate indicated to be in store for the Ohio River in the foreseeable future, probably within the time required to fully complete the modernization program.

In fact, at one point—Locks and Dam 41 at the Falls of the Ohio at Louisville—it will be virtually a race with time, beginning in 1957 with reconstruction of the facilities, to complete the job before the physical capacity of the present locks will have been reached some 5 or 6 years hence. This particular restriction results from a combination of a narrow, two-mile canal approach, a relatively high lift of 37 feet, and slow lock filling and emptying characteristics. These factors establish the physical capacity of the Louisville locks at an annual rate of about 19 million tons. However, at other locations on the Ohio where some of the restrictions present at Locks and Dam 41 are absent or are less severe, the physical capacity of the locks is substantially greater. From the results of a lockage-time study made during the last four months of 1953, assuming a reasonable allowance for increased delays as the saturation point is approached and a liberal allowance for increase in average loading, it is estimated that the practical physical capacity of most of the locks is at an annual rate of not more than 45 million tons.

The maximum annual tonnage to date (1956) past any Ohio River lock was attained in 1955 when almost 20 million tons moved past the locks in the reach of river between Huntington and Cincinnati. Thus, it would appear at first glance that a reserve capacity for the future of some 25 million tons would leave little cause for concern. But when the record of growth and the future prospects are analyzed the remaining reserve capacity begins to appear almost ephemeral. Figure 1 provides the basis for analysis of the traffic patterns and growth trends on the Ohio River.

The principal chart in Figure 1 depicts the changing magnitude, but similar pattern, of traffic distribution throughout the length of the Ohio River at five-year intervals, beginning in 1930, the year following completion of the existing lock and dam system, and ending in 1955. The horizontal scale represents the 981-mile length of the river from Pittsburgh to the mouth near Cairo. Each grid of the vertical scale represents one million tons. The following characteristics of the 1930 traffic will be noted:

From the mouth of the river to about Cincinnati, the traffic was fairly uniform at an average rate of under one million tons.

From Cincinnati to Huntington there was a marked increase to an average level of about 3 million tons.

From Huntington to above Wheeling the average tonnage fell to a level about equal to that below Cincinnati.

From above Wheeling to Pittsburgh there was a gradual but sharp increase to about 11 million tons at Pittsburgh, reflecting mainly the long-established coal traffic to steel mills in the upper Ohio Valley.

In 1935 it will be noted that the tonnage distribution is comparable to that for 1930 with virtually the same traffic pattern and that, except in the extreme upper end of the river, there was a small increase in tonnage throughout despite the economic depression at that time.

For 1940 the chart shows virtually the same traffic pattern as that for prior years and an average tonnage level two or three times that indicated for 1930 and 1935, again excepting the extreme upper river area where the increase was not so sharp.

The 1945 and 1950 graphs, as shown on the main chart, demonstrate the retention of the earlier pattern of traffic, but show the very substantial growth throughout the length of the river amounting to an 8-fold increase in 1950 in the reaches between the mouth and Cincinnati, and between Huntington and Wheeling, and to a 4-fold increase between Cincinnati and Huntington.

The graph for the calendar year 1955 presents striking evidence of the continued trend in traffic growth on the Ohio. It is striking not only in the magnitude of the absolute values of increase in the last five-year period, but also in demonstrating the continuing trend of acceleration in rate of growth that has been shown by records for preceding years. It also indicates some change in the traffic pattern and traffic distribution.

The smaller comparison charts shown in Figure 1 present mute evidence of the increasing trend in traffic growth in tonnage and ton-miles from 1915 through 1955. It will be noted that these charts indicate a continually accelerating growth during the 41 year period.

Suffice it to say then, that prudent planning calls for assumption of the necessity for completing a modernization and replacement program as rapidly as practicable, preferably by no later than 1970.

Major Criteria for Program

What, then, should be the major criteria or principles governing the formulation of the modernization program? Within the broad concept of providing a system of maximum efficiency for the user and of minimum costs to the operator for maintenance and operation, certain basic factors emerge. Obviously, fewer locks and dams with locks of adequate size will reduce travel time and tow operating costs. So will deeper pools impounded by higher dams, through the reduction of resistance to propulsion in deeper water. Fewer structures also will decrease the number of elements that must be maintained and operated, and modifications in design to eliminate repetitive operations will reduce manpower required at each lock and dam. The application of these criteria results in a major departure from the initial concept of movable dams with elements that can be lowered to the bottom of the river when stages are equal to or greater than the designed minimum channel depth. Some of the factors involved in such a major departure warrant examination.

First, the preponderance of downbound loaded tows is a thing of the past. The facility of moving that type of traffic on an open river, with the dam in lowered position, was the prime virtue of the wicket dam. With that advantage for the wicket dam eliminated, consideration can be given to higher dams of types which inherently prohibit "open-river" navigation.

Once that change is made, it is desirable to go to a dam as high as may practically be developed because the higher the dam, the fewer locks and dams will be required—thus serving the interests of the user through greater efficiency in towing operations, and serving the interests of the operator through lowered costs for maintenance and operations. But,of course, there are practical limitations on the height of dam that can be supported economically.

Generally speaking, the normal pools impounded by the dams must be kept within the banks of the Ohio. The reason for this will be recognized by anyone familiar with the high state of development of the valley floor. Great industrial developments, major transportation routes, and urban centers are liberally distributed throughout most of the upper three-fifths of the valley above Louisville; and lesser concentrations of similar improvements but much wider expanses of agricultural lands occupy the valley below Louisville.

Within this limitation, then, the characteristics of the Ohio River channel have a decided bearing upon practical heights of dams. Again, speaking generally, it may be stated that above Louisville the channel is relatively deeply entrenched, except for the upper 25 miles in the Pittsburgh area. Impoundments in the range of 19 to 35-foot vertical steps, at intervals permitting establishment of a minimum channel depth of 9 to 12 feet, can be retained largely within banks. Most of the steps in this reach will be in the 20 to 25-foot range. Below Louisville, the shallower entrenchment limits within-bank impoundments to about a 20-foot vertical step range.

Having established the approximate limits of heights of impoundments, the next controlling criterion—and a most important one—is the choice of locations for individual locks and dams which will most satisfactorily meet the requirements of navigation from the standpoint of ease and safety of approaching the locks. The site for the locks should provide not only for good alignment from both upstream and downstream directions, but also for good hydraulic conditions—that is, for the magnitude and direction of current velocities in the approaches that will facilitate the entrance and exit of traffic. Sites with desirable navigation characteristics are not plentiful and selection must be made with care. Model studies of current patterns in lock approaches are being used with good effect in selecting sites and structure arrangements; and in some cases have pointed the way to considerable improvement in approach conditions. A lesser factor, but one which is applicable to the extent that the merits of lock approaches are not sacrificed to any appreciable degree, is the selection of the site presenting the most favorable foundation conditions. Rock is not available for structure foundations in many reaches of the Ohio. However, there are areas where the shift of a site a few miles will mean the difference between a rock foundation or a pile foundation for all or part of a given structure. There is usually substantial economy in a site for structures which provides a rock foundation at favorable elevations.

The Program Plan

After consideration of all the factors mentioned heretofore, an overall plan of modernization and replacement of existing obsolescent facilities on the Ohio has been evolved. The general location of proposed structures is shown in Figure 2 in plan and profile.

Parts of the plan have been definitely fixed on the basis of detailed planning—witness the fact that new locks and dams are under construction at Markland, Greenup, and New Cumberland and the reconstruction of Locks and Dam No. 41 at Louisville will be initiated before July 1957. Other parts of the plan are fixed as to type of structures, but the exact site locations and precise pool elevations are tentative pending more detailed study. This is true for the remainder of that part of the plan not yet under construction above Louisville, where it is known that gated, non-navigable dams are the proper solution in lieu of the existing wicket dams, but some of the exact sites and pool elevations remain to be definitely established. For that part of the plan below Louisville, further study is indicated to determine the type of dam as well as definite structure locations and pool elevations. This is true because the maximum height of dam which appears practicable in the lower river is in the range of the upper limit of height adaptable to wicket operation. The choice of dam thus becomes a matter of balance between economy and desirable navigation characteristics. Both economy and navigation advantages are obviously in favor of the relatively high, gated dam above Louisville, but the choice is not so obvious one way or the other in the lower river.

It may be noted in passing that the location of structures for the proposed plan of replacement was fixed in the beginning at several points. For example, the elevation of the pool behind Locks and Dam 41 at Louisville is already about as high as practicable. The same is true at Gallipolis Locks and Dam where the same general type of dam now proposed for all the upper river was completed in 1938 as a replacement for several old locks and dams on the Ohio and Kanawha Rivers; and at the three locks and dams immediately below Pittsburgh, which also are replacements for earlier wicket dams.

There are other criteria of a general nature which must be considered carefully in designing the elements of the plan.

Criteria for Dams

In a valley with a history of floods as devastating as those of the Ohio, no artificial obstructions should be created that will materially add to the flood problem. The navigable wicket dam, of course, met this problem admirably. With its sill essentially at stream-bed level, and with all the wickets lowered when river stages are far below flood level, it constitutes a minimum of obstruction to flood flows. For the change to a relatively high, non-navigable type dam, it is apparent that damming elements which may be removed as flows increase will result in the least obstruction to flow. The answer to this problem is a gated dam with the gates surmounting a low sill, the crest of which is only a few feet above stream bed. Such dams have been in use for a number of years in the upper Mississippi and upper Ohio Rivers. Figure 3 is a photograph of the existing Gallipolis Dam on the Ohio above Huntington, which exemplifies this type. By adopting this type dam and

locating it in a suitable river section, the swellhead created at the dam by the sill, piers and lock structures can be held under one foot. This head is dissipated within a short distance upstream. The swellhead can be made even less by designing and constructing the lock facilities so that a part of the river's flow may be passed through the lock chambers when stages are reached at which traffic no longer can use the locks.

Another general criterion applying to the gated dam is the necessity for clearing maximum river stages with the gates in fully raised position in order to avoid the hazard resulting from trapping debris and large floating objects which may be expected with high stages. On the Ohio the criterion has been adopted of providing at least five feet of clearance between the maximum stage of record and the bottom of the gates in fully raised position. Since the range of stage is so great on the Ohio—amounting to as much as 80 feet between extreme low and high water in the Cincinnati area—the piers must be quite high.

A related criterion for the dam is at least partly responsible for adoption of a minimum gate length of 100 feet. This length reasonably assures passage of debris between piers so that accumulations do not occur which may cause both adverse effects from damming action and also obstructions which would prevent closing the gates as the river stages recede.

Still related to provisions for passage of debris, and ice as well, is a general criterion which allows for partial submergence of the gates, to a depth of about seven feet below normal pool elevation, in order to skim ice and debris over the top of the gates at times when river stages are not high enough to permit raising the gates clear of the water surface.

While upon the subject of the dam, it may be of interest to consider briefly the composition of the non-navigable type which is already being used to some extent and is proposed for extensive use in the replacement system. The characteristics of the low sill and high piers for this type dam already have been mentioned. Any of several types of gates would be workable for the sizes involved.

Already in use at Gallipolis Dam in comparable size is the "roller" type gate. The cylinder, with a diameter approximately equal to the damming height, is rolled up an inclined rack set in the piers. The hoisting devices are located on the top of each pier. Thus, the piers are required to be of sufficient height to allow for the height of maximum high water above normal pool, plus a minimum freeboard of five feet, plus the full diameter of the gate when in fully raised position.

Also presently in use on the Ohio at Emsworth and Montgomery Dams is the vertical lift gate. The gate at Emsworth is a framed, rectangular section which is lifted vertically between guides recessed in the piers. The hoisting machinery for these gates also is placed atop the piers. The gate in use at Montgomery is framed differently since it is built with three chords but operates as a vertical lift gate. The height of piers required for these gates is established by the same factors involved in the case of the roller gate.

A third type of gate used on the back channel portion of Emsworth Dam is a tainter gate so arranged that after the gate is pivoted on its trunnions to an open position clear of the water surface, the whole gate can be lifted vertically, with the trunnions following guides slotted in the piers. This gate is known as the Sidney gate, having been designed by Mr. W. E. Sidney when he was employed in the Pittsburgh District of the Corps of Engineers. It, too, requires hoisting machinery surmounting the piers and requires approximately

the same clearances and pier heights as do the roller and vertical lift gates.

All these types of gates have been considered for the proposed non-navigable replacement dams. There is also under consideration at the present time a conventional tainter gate. Design studies and investigations made thus far indicate this gate may be the most desirable and economical of all. The studies indicate that the trunnion sizes and bearing pressures can be kept within tolerable limits. Long trunnion arms will be required to rotate the gate through the arc necessary to clear maximum high water but, even so, the piers are indicated to be more economical than for the other types of gates because they can be made somewhat thinner and substantially lower. Model studies of this gate are being made, including a study of its adaptability to act as an overflow gate with submergence up to a maximum of about seven feet in order to pass ice and debris at normal pool stage.

Criteria for Locks

The criterion of providing two parallel lock chambers at each non-navigable dam already has been put in practice at the existing non-navigable dams on the Ohio River, and for good reason. Inasmuch as all traffic must pass through the locks in such cases, and has no alternative possibility of by-passing the locks as is true for a substantial part of the time in the case of the low navigable dams, a second chamber is imperative if reasonably dependable service is to be assured when the first chamber is closed for ordinary or emergency maintenance and repair, particularly on a waterway as heavily used as the Ohio. In fact, traffic on the Ohio has grown to the point that, should some of the future replacements include navigable dams—for example, in parts of the river below Louisville—two parallel lock chambers undoubtedly will be warranted at those locations also. Of course, two lock chambers at each site also have the advantages of providing an auxiliary to expedite traffic when there are several arrivals at about the same time, and of enhancing the physical capability of passing large volumes of traffic that may develop in the future.

The matter of establishing the size of locks best adapted to a waterway and its traffic is a study in itself. A paper dealing exclusively with this subject was presented before the American Society of Civil Engineers at a Pittsburgh meeting in April 1948 by Mr. R. L. Bloor of the Office, Chief of Engineers, Corps of Engineers. The matter is alluded to here only to fill out the picture on the plan of modernization for the Ohio and not with any attempt to develop the background leading to the choice of size. It is sufficient to say that the size of the principal lock chamber at each site—110 feet wide and 1,200 feet long—is considered to be the maximum for which reasonably consistent demand can be foreseen because of the characteristics of the river and of the equipment expected to use it. This chamber is the same width and twice the length of existing locks on the Ohio. As a matter of judgment, an auxiliary chamber at each site equal in size to existing locks—that is, 110 feet wide and 600 feet long—is considered to provide reasonably adequate capacity to assure dependable service for the lengths of time the main chamber can be expected to be out of service.

The question of height of lock walls is of much greater moment in the case of non-navigable dams than in relation to navigable dams since, in the case of the former, all traffic must go through the locks. No set criterion has

been used in establishing the height of lock walls for the replacement structures under construction. It was considered that any criterion which would relate the height of lock wall to an average outage of so many days per year would not be consistent or realistic because it does not define the length and frequency of separate outages. On the other hand, it is clearly uneconomical to think of building lock walls above maximum flood levels since along much of the river a few outstanding floods exceed all others by as much as ten feet. Moreover, all commercial traffic on the Ohio ceases when stages are considerably under maximum level. Accordingly, the characteristics of each project under construction have been examined and weighed in deriving the height of lock wall. Starting with a datum sufficiently low to be indicative, all flood hydrographs exceeding that datum were plotted at each lock site for periods of record up to 68 years. Generally, by examining these hydrographs a level could be chosen, by a variation in elevation of only a few feet, which would mean the difference between a large number of indicated recurrences at the lower limit as against very few recurrences at the higher level. In such cases the higher level would be chosen in view of the materially greater dependability of service provided at relatively small increase in cost. By such procedures, for example, the lock walls for the Markland project, now under construction, are designed to be 11 feet above normal pool. Based on 40 years of past record, this level is indicated to have been exceeded 5 times, with a maximum duration for any one time of 14 days and ranging down to a minimum duration of only 3 days.

Because of the increased size and lift of many of the replacement locks, very large volumes of water are involved in filling and emptying the main chamber. For example, to fill the Markland main lock from normal lower to normal upper pool level, more than 100 acre-feet of water must be let into the lock from the upper pool, and to empty it the same volume must be discharged to the lower pool. In order to handle traffic efficiently and expeditiously, the main chambers are designed to fill in about 8 minutes and to empty in about the same time. Thus, the average rate of filling and emptying is some 9,600 c.f.s. and the peak rate is in excess of 17,000 c.f.s. The filling and emptying system in the existing locks utilizes ports in the lock walls. Such a system would create intolerable turbulence and excessive hawser stresses for tows tied in the new main lock chambers, except for the lower lift structures, because of the much higher rates of filling and emptying involved in the higher lift structures. Consequently, a system has been adopted which provides parallel lateral culverts across the lock floor, with ports in the sides of the laterals spaced alternately in adjacent laterals. The laterals connect with culverts in the lock walls, and have been carefully positioned in the chamber on the basis of model studies.

No attempt is made in this paper to describe or discuss all of the details of the lock design. However, two features of the design warrant special attention. First, the arrangement of the bottom lateral filling and emptying system is unique in this country. Main lock design for the new high-lift locks on the Ohio River locates one group of laterals in the center of the upstream half of the lock. This group is fed by a culvert in the upper half of the river wall. This arrangement permits the discharge outlet for the upper group of laterals to be located near the center of the lock and shortens the length of the culvert which would otherwise be required in the river wall. A second group of laterals is located in the center of the downstream half of the main lock. This group is fed by a culvert located in the intermediate wall between

the main and auxiliary locks. The discharge outlet for the lower group is carried from the lower end of the culvert in the intermediate wall across the bottom of the main lock and through the river wall to discharge in the river. The idea for this design of the filling and emptying system, which splits the laterals into two separate groups, is of French origin, but the details of best adapted location of the two groups within the lock chamber have been worked out by exhaustive model tests made for the high-lift Ohio River locks. The results of the tests indicate that the hawser stresses developed when filling a 1,200-foot lock in eight minutes will be very little, if any, greater than those which would result from filling a 600-foot lock of equal lift in the same time. In this connection, a bottom lateral system is also used in the auxiliary lock, with a single group located near the center of this chamber. It is fed entirely from a culvert in the land wall. The lock discharge outlet extends from the lower end of the culvert, across the auxiliary and main lock chambers and through the intermediate and river walls to discharge in the river.

Second, the emergency gates for the higher lift main and auxiliary locks are unique in that submergible double-leaf lift gates are used for this purpose. The gates are designed to be operated under maximum head and also under conditions of unrestrained flow through the lock chamber. When not in use both leaves are lowered to prepared beds behind a concrete sill across the upper lock entrance above the upper miter gates. Appropriate hoisting equipment is provided for operation of the emergency gates. These gates have been tested in a model and results appear to be satisfactory. They will permit positive and rapid control in event of emergency, and also will permit manipulation of the miter gates during flood stages to provide for utilization of the lock chamber to pass unrestrained flow.

The new locks just described, together with reduction of the number of locks from 46 to 21 throughout the length of the river, will greatly facilitate the handling of traffic. The larger locks—1200 feet long in lieu of the existing 600-foot lengths—will eliminate the double lockages which are now required in many cases at the existing locks. For example, assume that an 1150-foot tow, including the towboat, leaves Pittsburgh bound down the Ohio River for a Lower Mississippi River port. Disregarding delay time due to congestion, and applying the average of 1.16 hours for the double lockage at each of the existing locks, the aggregate lockage time required for that tow to pass through the present 46 locks would amount to 53.3 hours, exclusive of pool travel time. With the fewer, new 1200-foot locks, applying an estimated 30 minutes per lockage which can be effected in one operation at each lock, the aggregate lockage time required for the tow in traversing the length of the Ohio would amount to 10.5 hours, exclusive of pool travel time. Thus, the example tow would save 42.8 hours in lockage time alone in its one down river trip when the proposed new facilities are completed. The benefits inherent in this vastly improved operation are apparent, aside from the realization of physical capacity adequate to handle a volume of traffic which promises to outgrow present facilities within the foreseeable future. In this connection the new locks are expected to more than triple the capacity of existing facilities.

In summary, modernization of the Ohio River navigation facilities is being accomplished, by reconstruction and replacement, to provide the fewest number of locks and dams consistent with the character of the channel, the state of development of the valley, and the costs associated with these factors.

Every feasible effort is being made to design facilities which will serve navigation efficiently and at the same time reduce operation and maintenance requirements and costs to a minimum.

Summary of Ohio River Plan

Referring once more to Figure 2 which depicts the modernization plan, still tentative in part, its features may be summarized, beginning at the head of the river at Pittsburgh:

Reconstruction will be accomplished at the first three locks and dams—Emsworth, Dashields, and Montgomery—principally by new lock construction at each location to provide facilities equivalent to those being built downriver. Present dams probably will be retained. Consideration will be given, however, to the advisability of a small increase in height of Dashields Dam.

From Montgomery Locks and Dam to existing Gallipolis Locks and Dam, 17 existing wicket dams and the single 110-foot by 600-foot lock chamber at each will be replaced by 6 non-navigable, gated dams with 2 parallel lock chambers of the size and characteristics previously described at each.

At Gallipolis Locks and Dam, a third chamber 110 feet by 1200 feet will be added landward of the present locks and on an improved alinement. The dam will be retained.

Between Gallipolis Locks and Dam and Louisville Locks and Dam, 13 existing wicket dams and their locks will be replaced by three non-navigable, gated dams and their dual modern locks previously described.

At Louisville, the existing auxiliary 56-foot by 360-foot lock will be reconstructed to provide a modern 110-foot by 1200-foot chamber, retaining the existing 110-foot by 600-foot lock as an auxiliary. The present narrow, two-mile canal forming the upper approach to the locks will be modified to provide an unrestricted approach. A major part of the existing dam will be modified and retained and a new, non-navigable gated section similar to the proposed dams above Louisville will be added for regulatory control over the pool.

Tentatively, from Louisville to the mouth of the river the 11 existing wicket dams and 110-foot by 600-foot locks will be replaced by 7 new dams with two modern locks at each. Final determination of the type of dam and number to be provided in this reach remains for further detailed study.

Tributary Programs

This paper deals mainly with modernization of navigation facilities on the Ohio because of its significance as the major artery among the basin's navigable streams. However, several of its tributaries are of commanding importance as waterways in their own right. These active, canalized tributaries comprise the Allegheny, Monongahela, Kanawha, Green, Cumberland, and Tennessee Rivers.

Of these six tributaries, the Allegheny and Kanawha have facilities which are in good structural condition and which are reasonably adequate to handle the volume of commerce utilizing these waterways. Larger lock sizes may well be in order on the Kanawha if the traffic, amounting to 7.6 million tons in 1955, grows substantially in the future. With one notable exception, the Tennessee River also has reasonably adequate facilities. A bottleneck

created by three small locks in series at Wilson Locks and Dam is in the process of being eliminated by construction of one large modern lock by the Tennessee Valley Authority. On the other three tributaries—the Monongahela, Green and Cumberland—steps have been or are being taken to replace and reconstruct obsolescent, inadequate facilities.

On the Monongahela River, where traffic aggregated 37.6 million tons in 1955, progress is being made in the modernization program. In the 30 miles of the river below its head at Fairmont, West Virginia, the original system provided a minimum depth of only 7 feet, controlled by 6 dams with one 56-foot by 182-foot lock at each. An authorized replacement program comprising three new locks and dams for this reach has been partly completed. Morgantown Lock and Dam has been built at the lower end of this reach and Hildebrand Lock and Dam, the next upstream, is under construction. The lock size in this reach, 84 feet wide and 600 feet long, has been adapted to the characteristics of the channel, and to the type of equipment and character of traffic expected to use this section of the river. The uppermost new lock and dam, Opekiska, has not been started.

Below Morgantown Locks and Dam on the Monongahela, plans are being prepared to modify and raise Dam 8, next downstream, to assure a minimum 9-foot depth throughout that pool and thus complete the 9-foot channel throughout the length of the river. Preliminary plans are nearing completion for replacement of present inadequate facilities between Locks and Dams 8 and 4 where traffic is extremely heavy. At Locks 5, for example, there are two parallel 56-foot by 360-foot chambers. These locks have handled an annual volume in the order of 25 million tons which is virtually the limit of their practical capacity. Locks 4 and 3 each have a 56-foot by 720-foot chamber and an auxiliary 56-foot by 360-foot chamber. These facilities are satisfactory for the present. Locks 2, the lowermost on the Monongahela, have been rebuilt in recent years to provide one chamber 110 feet wide and 720 feet long and an auxiliary 56-foot by 360-foot chamber. Completion of the work in progress and in various stages of planning will give this river modern facilities consonant with its present and potential use.

On Green River modernization has been accomplished for the lower 100 miles of channel. Locks 1 and 2 and Dam 2 have been reconstructed and the locks were opened to traffic in mid-1956. Associated channel work is nearing completion. Each of the new locks was built on the abutment side of the dams to replace the more than 100-year old chambers which were only about 36 feet wide and 140 feet long. The size of the new locks, 84 feet wide and 600 feet long, is adequate for the character of the channel and the traffic expected to use it. No change was made in pool elevations which already averaged nearly 50 miles in length. It is anticipated that large volumes of coal and other commodities can be handled efficiently and economically by the new facilities.

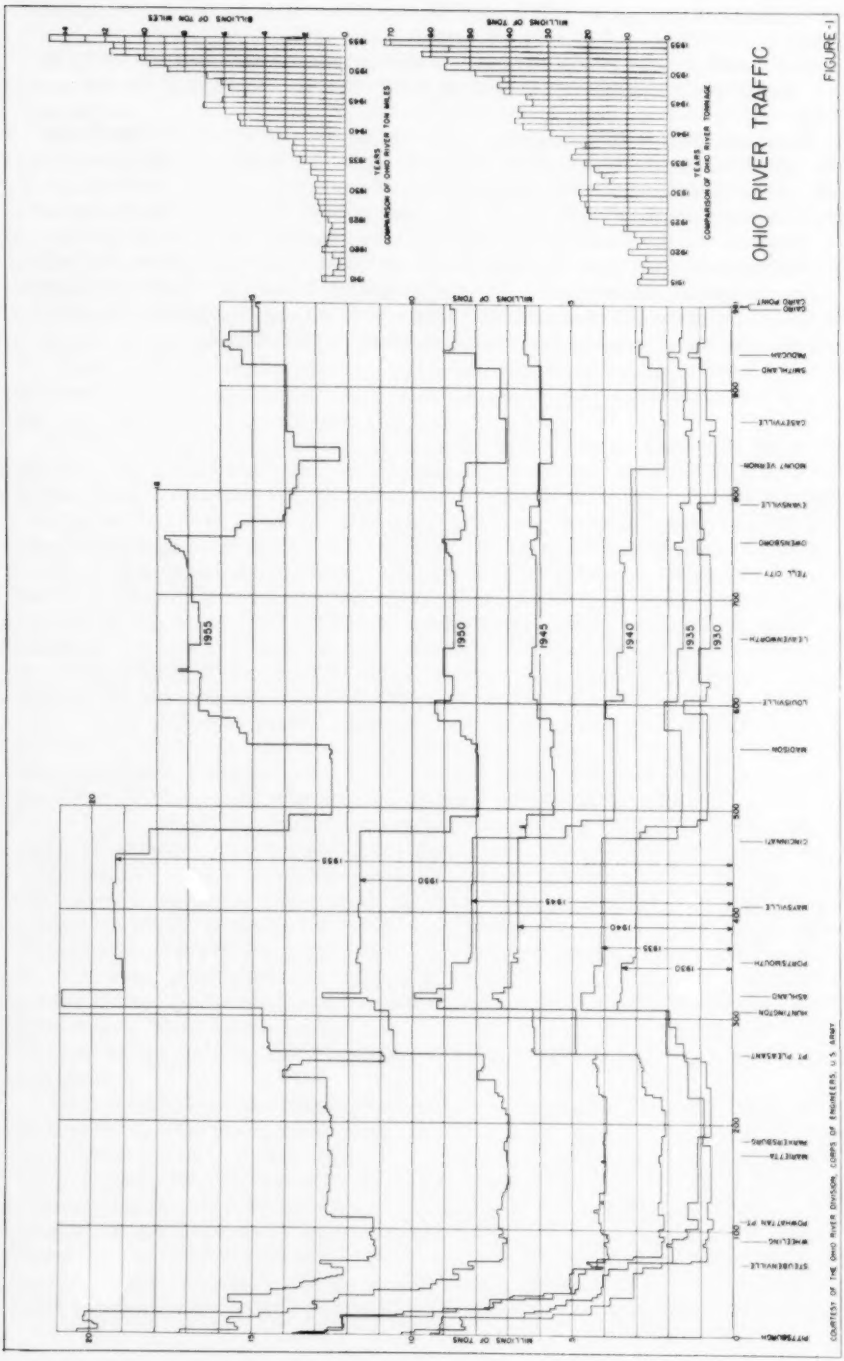
Modernization of navigation facilities on the Cumberland is far advanced. New locks at Cheatham, below Nashville, and at Old Hickory, above Nashville, have been opened to traffic and will be fully effective as soon as associated construction has progressed to the point of permitting the pools to be fully raised. These two new locks will replace 8 old, small locks between Cheatham and Carthage, Tennessee, some 300 miles above the mouth of the Cumberland. The remaining obstacle to navigation will then be the five small locks between Cheatham and the mouth of the river. This remaining bottleneck will be removed upon completion of Barkley Lock and Dam, which is

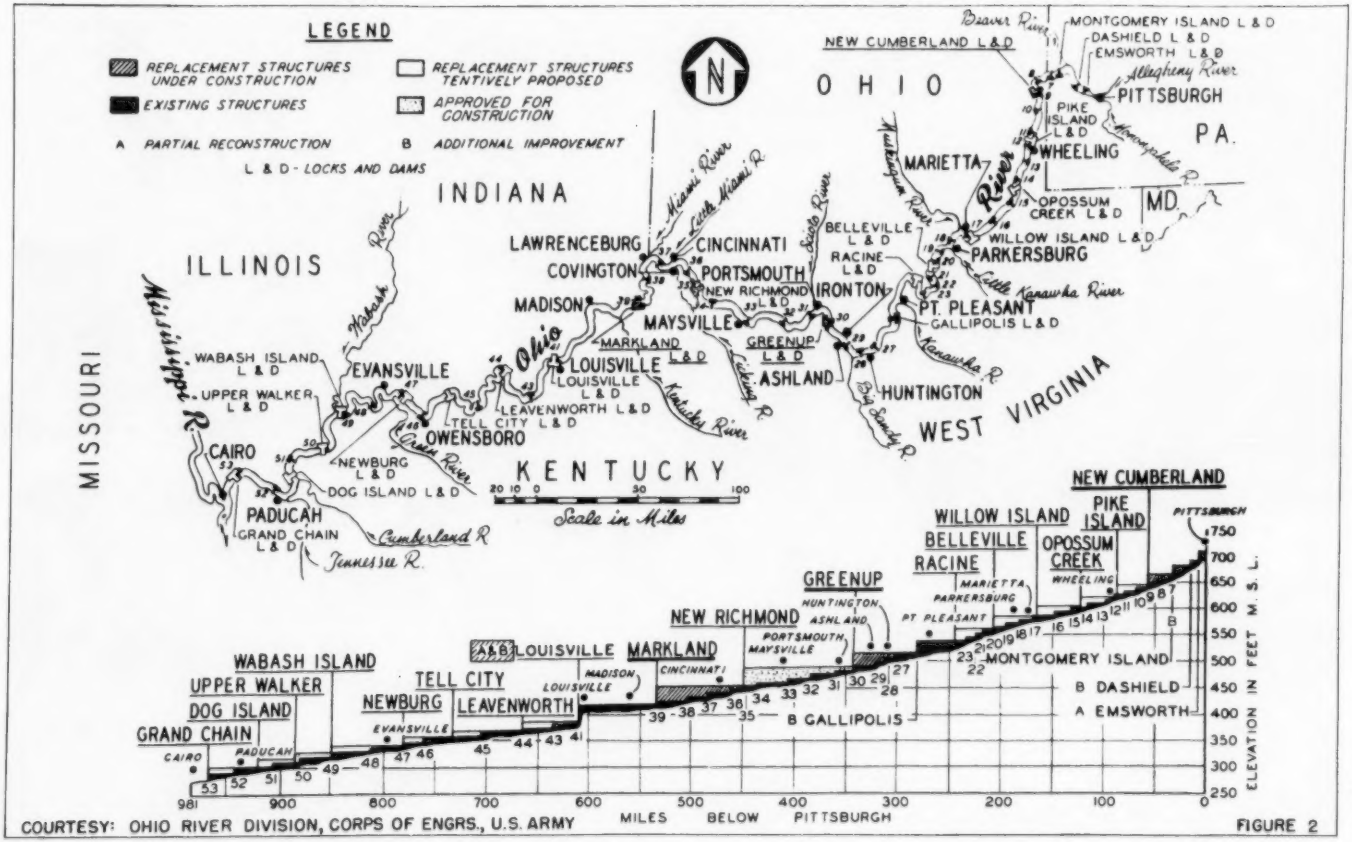
being placed under construction in 1957 at a point about 30 miles above the mouth.

The Barkley project will have the same operating levels and characteristics as the nearby Kentucky project on the Tennessee River, and the two pools will be interconnected by a short canal.

Thus, aside from their utilization for hydroelectric power and flood control, the three projects—Barkley, Cheatham and Old Hickory—will provide efficient, modern navigation facilities for 300 miles of river. The locks at Barkley and Cheatham will be 110 feet wide and 800 feet long, and the lock at Old Hickory will be 84 feet wide and 400 feet long, reflecting the difference in character of river and expected traffic demands below and above Nashville.

Completion of all the modernization programs described, on the Ohio and its active navigable tributaries, will keep the waterway transportation system abreast the constantly increasing demands made upon it.


OHIO RIVER TRAFFIC
FIGURE -1
COMPARISON OF OHIO RIVER TONNAGE
COMPARISON OF OHIO RIVER TON MILES
YEARS
MILLIONS OF TONS
BILLIONS OF TON MILES
1955
1950
1945
1940
1935
1930
CAIRO POINT
PADUCAH
SMITHLAND
EVANSVILLE
MOUNT VERNON
CRAWFORDSVILLE
OWENSBORO
TELL CITY
LEAVENWORTH
LOUISVILLE
MADISON
CINCINNATI
MAYSVILLE
PORTSMOUTH
ASHLAND
HUNTINGTON
PT. PLEASANT
MARIETTA
PARKERSBURG
POINT PLEASANT PT.
WHEELING
STEUBENVILLE
PITTSBURGH
COURTESY OF THE OHIO RIVER DIVISION, CORPS OF ENGINEERS, U.S. ARMY



COURTESY: OHIO RIVER DIVISION, CORPS OF ENGRS., U.S. ARMY

FIGURE 2

COURTESY: OHIO RIVER DIVISION, CORPS OF ENGRS., U.S. ARMY

FIGURE

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MISSISSIPPI RIVER - GULF OUTLET

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(Proc. Paper 1240)

SYNOPSIS

The Mississippi River-Gulf Outlet is to be a 36 ft. by 500 ft. navigation channel from the City of New Orleans, La., to the Gulf of Mexico, a distance of 76 miles. It will provide a slack water channel for ocean-going ships, reducing travel time of coast-wise vessels, eliminating the hazards of river travel, and affording expansion of harbor facilities with construction of more efficient wharves and appurtenances.

Pre-construction planning, which began this year, will allow channel construction to begin in July 1958. Depending upon availability of additional funds, five years will be required for construction.

This paper is a resume of the history and background of a project of vital importance to the City of New Orleans and the entire Mississippi Valley. It discusses the plan of development and costs of the project. Limited discussion of technical and engineering features is provided, but due to the present state of planning, the paper is essentially non-technical in content.

INTRODUCTION

The City of New Orleans, La., is located on the Mississippi River approximately 115 river miles from the Gulf of Mexico. The main portion of the City lies on the east bank of the river in a great bend which forms a crescent from which the City derives its title, "The Crescent City." Due to rapid growth in the last 25 years the City has extended beyond the original crescent shape. The growth, however, has been concentrated principally on the east bank of

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the river. A recent aerial picture of a part of the City and its east bank wharves is shown in Figure 1. The City, according to the 1950 census, has a population of 570,444 within its corporate limits and a metropolitan population of 685,405.

Being located near the mouth of the mighty Mississippi and its tributaries, New Orleans is the natural gateway to the Mississippi Valley and since early colonial times has been an important world port.

From these early times the wharves were built on the natural levees adjacent to the river. These levees were progressively raised to prevent flooding of the City as the levee system along the Mississippi developed. By 1915 the wharves and adjacent warehouses extended practically continuously for 7-1/2 miles along the east bank of the river. The wharves, extending from the levee to deep water in the river were built on long wood piling. Due to the variations of approximately 20 feet in the water level in the Mississippi River at New Orleans, as shown in Figure 2, the floor of the wharves had to be built very much higher than the natural ground to be a safe distance above the maximum high water. This difference in elevation is not conducive to economical loading and unloading of ships and operation of the necessary rail and other facilities.

About 1915 the need for additional harbor frontage to take care of the growing port business, as well as the need to serve industries whose economical operations depend upon the availability of water transportation and nearby homes for their employees, became apparent. At this time the port authorities adopted a long-range plan of development which included the construction of the Inner Harbor Navigation Canal and Lock, shown in Figure 3. The lock at the canal entrance to the river, built in the period 1918-1923, is 640 feet long, 75 feet wide and has a depth of 31-1/2 feet over the sill at low water. The canal essentially has a depth of 30 feet, varies in width, and extends approximately 5-1/2 miles from the river to Lake Pontchartrain.

The land purchased for the Inner Harbor and Navigation Canal included an area for establishment of water front industries, as well as slips for building additional wharves, with attendant railroad yards and other facilities. The establishment of slips and adjacent wharves, radically different from the past practice in New Orleans of placing wharves end to end paralleling the river, affords a means for compact development of wharves and railroad facilities to meet the modern methods of freight handling and warehousing. The variation of water level in the Inner Harbor and Navigation Canal, as shown in Figure 4, is insignificant compared to the variation in the river. This slight variation in water level permits wharves to be built at a much lower elevation than on the river, and is conducive to the economical loading and unloading of barges as well as deep draft vessels.

At the time of its conception, provisions were made for the eventual connection of the Inner Harbor with the Gulf of Mexico by means of a tide water channel. Such a channel would necessarily traverse extended stretches of undeveloped marsh and swamp land, automatically opening new territory for development of a modern port as growth required additional facilities.

Based on the preceding discussion it is evident that the Port of New Orleans consists of two harbors or areas for water-borne traffic,—the river front and the Inner Harbor. In general, the wharves along the river front are in excellent condition and will continue to serve as useful components of the port even though expansion is limited and the variations of water level and congestion in area are detrimental to efficient operation. Where practical the

wharves are being renovated and expanded to the fullest extent including a few new wharves in space available. The Inner Harbor, while expanding, is handicapped by delays attended in lockage and congestion in the vicinity of the lock. Elevation of the lock sill also makes use of the river wharves by the larger ships a necessity.

Authorization

The Division Engineer, U. S. Army Engineer Division, Lower Mississippi Valley, in compliance with instructions from the Chief of Engineers, and in response to Congressional resolutions, submitted to the Chief of Engineers a review report dated 30 September 1946, recommending the construction of the Mississippi River-Gulf Outlet as a modification of the existing project, Mississippi River, Baton Rouge to the Gulf of Mexico. The Board of Engineers for Rivers and Harbors, after considering the recommendations presented in the Division Engineer's report, information presented at a public hearing, and various other data, concurred in providing the outlet proposed by the Division Engineer. By letter dated 5 May 1948, the Chief of Engineers submitted to the Secretary of the Army for transmission to Congress the report of the Board of Engineers for Rivers and Harbors concurring, in general, with the views and recommendations of the Board. The report of the Chief of Engineers, after receiving favorable consideration by the Bureau of the Budget, was submitted to Congress on 25 September 1951 by the Secretary of the Army. The above described reports and letters were printed as House Document No. 245, 82nd Congress, 1st Session, entitled "Mississippi River-Gulf Outlet."

The Act authorizing construction of the Mississippi River-Gulf Outlet, approved by the President on 29 March 1956, reads as follows:

"Public Law 455-84th Congress, Chapter 112 - 2d Session, H.R. 6309. An Act, To authorize construction of the Mississippi River-Gulf Outlet. Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That the existing project for Mississippi River, Baton Rouge to the Gulf of Mexico, is hereby modified to provide for the Mississippi River-Gulf Outlet to be prosecuted under the direction of the Secretary of the Army and supervision of the Chief of Engineers, substantially in accordance with the recommendation of the Chief of Engineers contained in House Document Numbered 245, Eighty-second Congress, at an estimated cost of $88,000,000: Provided, That when economically justified by obsolescence of the existing industrial canal lock, or by increased traffic, replacement of the existing lock or an additional lock with suitable connections is hereby approved to be constructed in the vicinity of Meraux, Louisiana, with type, dimensions, and cost estimates to be approved by the Chief of Engineers: Provided further, That the conditions of local cooperation specified in House Document Numbered 245, Eighty-second Congress, shall likewise apply to the construction of said lock and connection channels. Approved March 19, 1956."

It will be noted that the Act also provides for the replacement of the existing Inner Harbor Canal Lock or additional lock and connections, near Meraux when economically justified.

A map of the authorized project is shown in Figure 5. Initial funds for advance engineering and design of the project were made available this fiscal year.

Purpose of Outlet

In order to obviate the necessity of lockage, to provide a slack water channel free of the hazards of flood stage river navigation and to provide for the continued orderly expansion or growth of the port, the interested local organizations and State agencies secured the necessary Congressional authorization to provide the Inner Harbor with an outlet to the Gulf as previously conceived many years ago.

The Mississippi River-Gulf Outlet provides a direct, controlled deep-draft navigable outlet to the Gulf from New Orleans, a route which will not be affected with sedimentation from the upper reaches of the river, which sediment has proven to be a detriment and impediment to navigation in the river.

Justification

To fully understand the reasons why port authorities and other navigation interests have shown such great concern in the authorization and construction of a seaway canal linking New Orleans to the Gulf of Mexico, it is first necessary to understand the reason for the tremendous increase in waterborne commerce to and from the Port of New Orleans during recent years and to realize the significance of this increase. In 1916, as indicated in Figure 6, the tonnage through the Port of New Orleans totaled 7-1/2 million tons. In 1944, during World War II, it was about 24 million tons, and in 1955 the total tonnage was over 47 million tons. Of this tonnage for 1955, 53 per cent was composed of petroleum products or petroleum crude, much of which originated in the fast growing oilfields in southern Louisiana and along the Gulf Coast. The bulk of the petroleum crude is destined for oil refineries on the Mississippi River and would probably not utilize the seaway canal. Other ships carrying bulk cargoes, such as sulfur, ores, and grain would likewise probably not use the seaway canal, due to existing special facilities on the river.

The vast majority of the remaining tonnage, consisting of general cargoes, are now handled by the wharves along the east bank of the river and in the inner harbor. These facilities are taxed to capacity, and quite frequently boats that would come to the port are diverted to other Gulf ports due to the congested conditions. The port authorities have been renovating the existing facilities enabling them to carry larger loads per square foot of area, adding additional sheds wherever practical and constructing new facilities on the river and inner harbor; however, the space for economical operations is practically all utilized. In order properly to expand with the continuing increase in traffic it is essential that additional deep sea wharf facilities for both general cargo and industrial sites be provided. This could best be done by the construction of a seaway canal connection to the existing inner harbor and navigation canal which recently exhausted its last major industrial site. In addition to the monetary benefits, which will be discussed later, the seaway canal will provide a second outlet to the sea which in a time of national emergency may prove to be of inestimable value. During the last war, both the Army and Navy made extensive use of the port facilities. Naturally, the possibility of immobilizing the port with two outlets and with facilities dispersed over large areas is much more remote than that which exists today with only the river as an outlet.

Benefits and Costs

The benefits to be derived from the project will occur principally from general cargo commerce. Petroleum interests have an aversion to the use of side outlets and passage through a lock and have stated that they will continue to use the river in lieu of the outlet. The other bulk commerce, such as sulfur and ores, already have specialized terminals on the river, outside the developed portion of the port, and will also continue to use the river. In 1946 at the time the Division Engineer's report was written, it was estimated that the prospective seagoing general cargo commerce pertinent to the outlet and east bank portion of the port may increase to more than 10,000,000 tons. A chart showing the increase in deep draft tonnage is shown in Figure 7. The Board of Engineers in their study used somewhat different postulates in arriving at the benefits to be derived from the project. They estimated in 1948 that commerce in general cargo alone would reach an annual volume of 7,500,000 tons in about 10 years. Of this amount it was estimated that 3,250,000 tons would move over new terminals to be provided in the Inner Harbor. It may be observed that those estimates were conservative, since the general cargo in 1955 exceeded 6,000,000 tons. Conservative estimates of annual benefits adjusted to current prices indicate an annual savings of $6,815,000.

These benefits consist of

Savings in ships' time	$3,981,000
Savings in cargo handling	1,041,000
Other benefits	1,793,000
Total	$6,815,000

To elaborate on these savings it is estimated that the 1,550 vessels expected to use the new tidewater facilities annually, with modern terminal equipment, would experience a savings in turn-around time of 1-1/4 days per call. In addition, relief of congestion at the existing general cargo wharves due to use beyond economical capacity and congestion of landward access facilities will save time in loading and unloading and provide benefits to industrial and commercial tonnage at an estimated savings of $1,041,000 annually. Further benefits credited to the improvement result in reduced sailing time of coastwise vessels, reduction in hazards to navigation, savings in terminal handling charges, annual charges on the upkeep of the facilities, and enhancement in waterfront property amounting to an additional saving of $1,793,000 annually. The current estimate for the project is $92,000,000 for Federal and $2,300,000 for non-Federal or local interests' costs. The Federal costs include excavation of the channel and turning basin, and construction of dikes and jetties and a highway bridge. The non-Federal costs represent procurement of rights-of-way for the channel and turning basin and for spoil disposal areas, but do not, however, include the cost of construction of new terminal or transfer facilities which will have to be provided by local interests as the Inner Harbor develops, nor do they include any annual operation and maintenance costs. The annual charges for the project amount to $4,573,000 (both Federal and non-Federal) including interest on gross investment, amortization, and operation and maintenance.

The ratio of annual benefits ($6,815,000) to annual charges ($4,573,000) gives the favorable Benefit/Cost ratio of 1.5 to 1 for the project. None of the

above figures include the costs of the additional lock and connections near Meraux which will require justification at the time construction becomes imminent.

Description of Improvements

The project consists essentially of approximately 76-1/2 miles of channel excavation, including an eased entrance at the Gulf, protective jetties and wing dikes at the entrance, a retention dike across Chandeleur Sound, a turning basin and a bridge across the channel near the New Orleans end of the project. The tentative location of the proposed channel, a seaway canal 36 feet deep and 500 feet wide, is as indicated in Figure 5. The channel commences at the intersection of the existing Gulf Intracoastal Waterway with the Inner Harbor Navigation Canal approximately 7500 feet north of the existing lock and approximately 11,000 feet from the Mississippi River. From this intersection the channel, utilizing a one-degree entrance curve, follows the existing Intracoastal Waterway in a generally easterly direction for a distance of 5.5 miles to a point approximately one-half mile below Highway 47 (Paris Road). The channel thence leaves the Intracoastal Waterway via a one-degree curve and continues approximately 63 miles in a southeasterly direction to and along the south shore of Lake Borgne and thence in a general southeasterly direction through the marshes to and across Chandeleur Sound to a point in the vicinity of Errol Island. From this point an eased entrance is to be constructed gradually increasing in width from 500 feet to 600 feet and increasing in depth from 36 feet to 38 feet. This eased entrance extends approximately eight miles to the -38 foot contour in the Gulf of Mexico. Protective jetties, extending out to the -20 foot contour in the Gulf from the Island, will be constructed on each side of the entrance, as well as suitable wing dikes on the island to prevent it from erosion by wave action.

The channel route crosses approximately 26 miles of open water in Chandeleur Sound. A retention dike which will protect the channel from the generally prevailing south and southwest winds and which will prevent the reentry of the spoil into the dredged channel will be constructed. It is contemplated that this dike, or dikes on both sides if necessary, will be constructed from spoil protected by stone on the faces.

Near the New Orleans terminus of the channel a turning basin 1,000 feet by 2,000 feet will be provided.

Also, near the New Orleans terminus a suitable bridge for State Highway 47 (formerly (61) will be provided. While negotiations have not begun with the Department of Highways, State of Louisiana, regarding the type clearances, design criteria and other features of the bridge to be built at this location, due to the width of channel and nature of traffic, the bridge will probably be of the semi-highlevel vertical lift type.

The above described features are generally in accord with the findings of the Board of Engineers for Rivers and Harbors and the Division Engineer. It is to be noted, however, that the Division Engineer made provisions for the future construction of a lock and connecting channels in the vicinity of Meraux, when such is found to be economically justifiable. It was anticipated by the Division Engineer that provision ultimately should be made, and would be justified economically, for a lock 760 feet long, 110 feet wide and with a depth of 40 feet over the sills, and the necessary channel connections between the river and the turning basin of the outlet. The authorizing Act as finally

adopted included the provision for a future lock at Meraux or a replacement lock on the Inner Harbor Navigation Canal. It may be stated that the inclusion of such a lock in the authorization was for purposes of future planning only. Monetary authorization for construction will require further justification to the Congress.

Technical Features

Soil borings are presently being made along the channel route to determine in detail the various types of soils to be excavated; to establish the side slopes for the proposed excavation; soil disposal limitations; and to estimate the amount of settlement that can be anticipated on the jetties and protective dikes. Natural wave action in the open water areas, as well as waves created by passing ships, will subject the channel slopes and protective dikes to severe wave action and erosion. The soil studies in conjunction with beach erosion studies will furnish information as to the need for protective pavements or revetments along the slopes of the spoils or dikes in the open water areas, and the slopes of the excavation in the marsh land areas.

From available geological information it is known that the region traversed by the proposed ship channel was at one time a sub-delta of the Mississippi River. The pronounced natural levee ridges along Bayou LaLoutre marks the position of the course of the river or main distributary channel which carried flood waters and sediments of the Mississippi to the region and formed this ancient sub-delta. At the height of its development, the sub-delta extended seaward of the present position of the Chandeleur Islands. Subsequent geological events, consisting of loss of sediment-laden floodwater from the region, and gradual subsidence and seaward tilting of the land mass, combined with wave action against the outer fringe of the sub-delta, resulted in the formation of the Chandeleur Islands, and the sunken sound between the Islands and the mainland.

Two principal geologic formations composed of sediments carried down and deposited by the Mississippi River comprise the subsurface of the area. From the ground surface to depths varying from about 46 feet at the New Orleans end of the channel to as much as 120 feet at the Gulf end, there is a formation of Recent geologic age composed predominantly of very soft gray clay with zones and strata of silt, very fine sand and peat. Generally, it is within 5 to 20 feet of the surface that the peat and highly organic clays exist. Beneath the Recent soils, there is a formation of Pleistocene age (the Prairie Formation) consisting of stiff to hard clays, fine sand, and silt. The soils of the Pleistocene formation are generally lighter in color and are much more compacted and firmer than the overlying very soft Recent soils. The channel will be excavated only in the Recent soils.

Other than preliminary soils explorations, studies to be made in connection with the design of the proposed jetties have not been initiated as of this date. Comparison of other jetties under the jurisdiction of the New Orleans District indicates that the characteristics of Calcasieu River jetties are probably the closest to those that may be constructed for the Outlet entrance. The Calcasieu River and Pass is a navigation project providing a 35 foot by 250 foot ship channel. The jetties on this project were originally constructed by local interests first of wood planks, piling, slab and creosoted timber. These were in turn destroyed by teredos (shipworms), storms and hurricanes. Several times the construction was abandoned, but later rebuilt. Construction

of the stone jetties was begun by local interests in 1892 and completed in 1899. Repairs and extension to the -12 foot contour by the Government was completed in 1943. Since that time no further maintenance has been required. These jetties are built on a blanket of riprap stone 2 feet thick and 100 feet wide, the stone weighing from 25 pounds to 200 pounds. The east jetty was constructed with a 10-foot crown 6 feet above mean low water and side slopes of 1 on 1-1/4.

The core of the jetty consists of rock varying from 25 pounds to 4 tons placed over the riprap blanket. Over the core a layer 4 feet thick of cover stone was placed, the cover stones weighing 5 tons to 10 tons each. If soft material is encountered in the base of the proposed outlet jetty a fascine willow mat will be placed under the riprap to aid in supporting the rock. The project contemplates construction of the jetties to the -20 foot contour in the Gulf. The jetties will probably be built outward in stages and it may be determined that the construction may not have to be carried all the way out to the -20 foot contour.

A network of hydrologic instruments or requirements are being installed or investigated to provide the basic data required to supplement existing meager information on tides, waves, wind directions and velocities, littoral currents, and salt and silt contents of the water. Locations where various types of data will be secured are shown on map of the general area, Figure 5.

These data will be utilized in project studies to provide estimates of the frequency of severe disturbances, force and effect of waves, movement of sediment, importance of tidal action, implications attendant to the overtopping of jetties and dikes, and other factors required for the engineering design and evaluation of the project.

Studies of past storms along the Louisiana-Mississippi coast indicate that maximum tides of from 10 to 15 feet may be expected. The various factors contributing to or attending such tides must be evaluated and specific design criteria developed for each factor or interrelated series of factors.

The possible influence of the deep channel or salinity problems in the interior is not being neglected. Salinity samples will be taken and analyzed for stations located in the vicinity of New Orleans, in the marsh near Hopedale and in Chandeleur Sound. Comparative before, during, and after construction salinity data should indicate and serve to evaluate any changes that may result from the construction of the project.

Plans of Port of New Orleans

Following completion of the Inner Harbor Navigation Canal, constructed by the Board of Commissioners of the Port of New Orleans, sites for industrial development and additional terminals became available. The Inner Harbor Navigation Canal and lock, which provides access for vessels not exceeding 30 feet in draft, affords several lateral slips, marginal wharves, two turning basins, and a connecting channel to the Intracoastal Waterway to the east. Of the multiplicity of sites along this 5-1/2 mile canal, all have been or are in process of being developed with the exception of a 12-acre tract and a 22-acre tract. Ability of the port authorities to further expand the port facilities along this canal is, then, prescribed by the near total utilization of space.

An opportunity for additional expansion will come with construction of the Mississippi River-Gulf Outlet. As an item of project participation specified in the authorizing Act, the local interests are required to construct, maintain,

and operate terminal facilities commensurate with requirements of the expanded port, under regulations approved by the Secretary of the Army, to ensure its equitable operation in the public interest. The Board of Commissioners, Port of New Orleans, has under active study and consideration the orderly planning and layout of the facilities to be constructed in accordance with the foregoing requirements.

Status of Work

The first work undertaken was to make reconnaissance soils borings over an extensive area in Chandeleur Sound to determine the nature of the soils to be excavated and their ability to support the proposed dikes and jetties. In order to more firmly establish the location of the channel through Chandeleur Sound, and because of the necessity of securing these data while weather conditions were favorable, this work was initiated as soon as funds were made available. Upon completion of the reconnaissance borings and analyses, a tentative line was adopted. A detailed survey of this line is now in progress. Additional borings are now being made on the proposed channel route. This work will permit determination of the channel side slopes, berm distances, and spoil requirements for the channel across the land portion of the outlet. The Beach Erosion Board has been requested to aid in the studies and to make recommendations regarding the type and locations of the dikes and jetties, as well as to evaluate the possible influence of hurricane wind waves propagated up the channel toward New Orleans. They are presently assembling available data on wind tides, wave heights, and current velocity and directions. Further data collection and studies on these features will be made this year and continued into next year to determine the effect of open water spoil disposal and influence on channel bottom movement induced by wave action and tidal currents. From these studies a determination will be made of the location to determine whether a retention dike will be required on only one or on both sides of the channel.

Salinity studies are also under way to determine the existing conditions along the route and the possible effects from excavating the deep channel.

Work on the various studies and preparation of plans and specifications for construction of the project (except the Highway Bridge) are scheduled for completion in fiscal year 1958. The construction work will require about 5 years to complete, however, the construction schedule will depend upon the availability of additional funds.

Completion of the Mississippi River-Gulf Outlet will provide an essential facility for the Port of New Orleans and the entire Mississippi River Valley for an orderly and continuous development consistent with the requirements of a rapidly expanding national population and economy.

Fig. 1. Aerial View, Portion of City of New Orleans and East Bank, Mississippi River Wharves

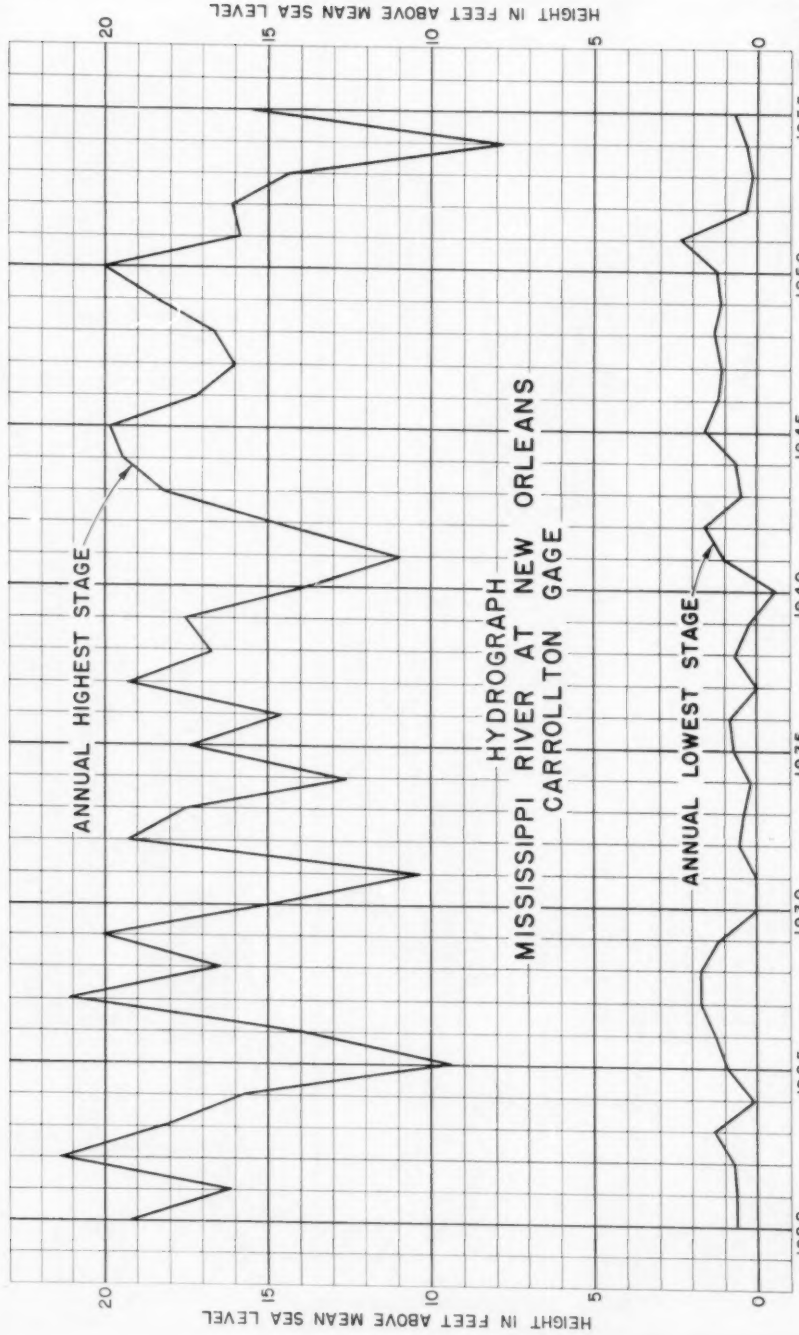

HYDROGRAPH
MISSISSIPPI RIVER AT NEW ORLEANS
CARROLLTON GAGE
ANNUAL HIGHEST STAGE
ANNUAL LOWEST STAGE
HEIGHT IN FEET ABOVE MEAN SEA LEVEL
0
5
10
15
20
1920
1925
1930
1935
1940
1945
1950
1955

Fig. 2

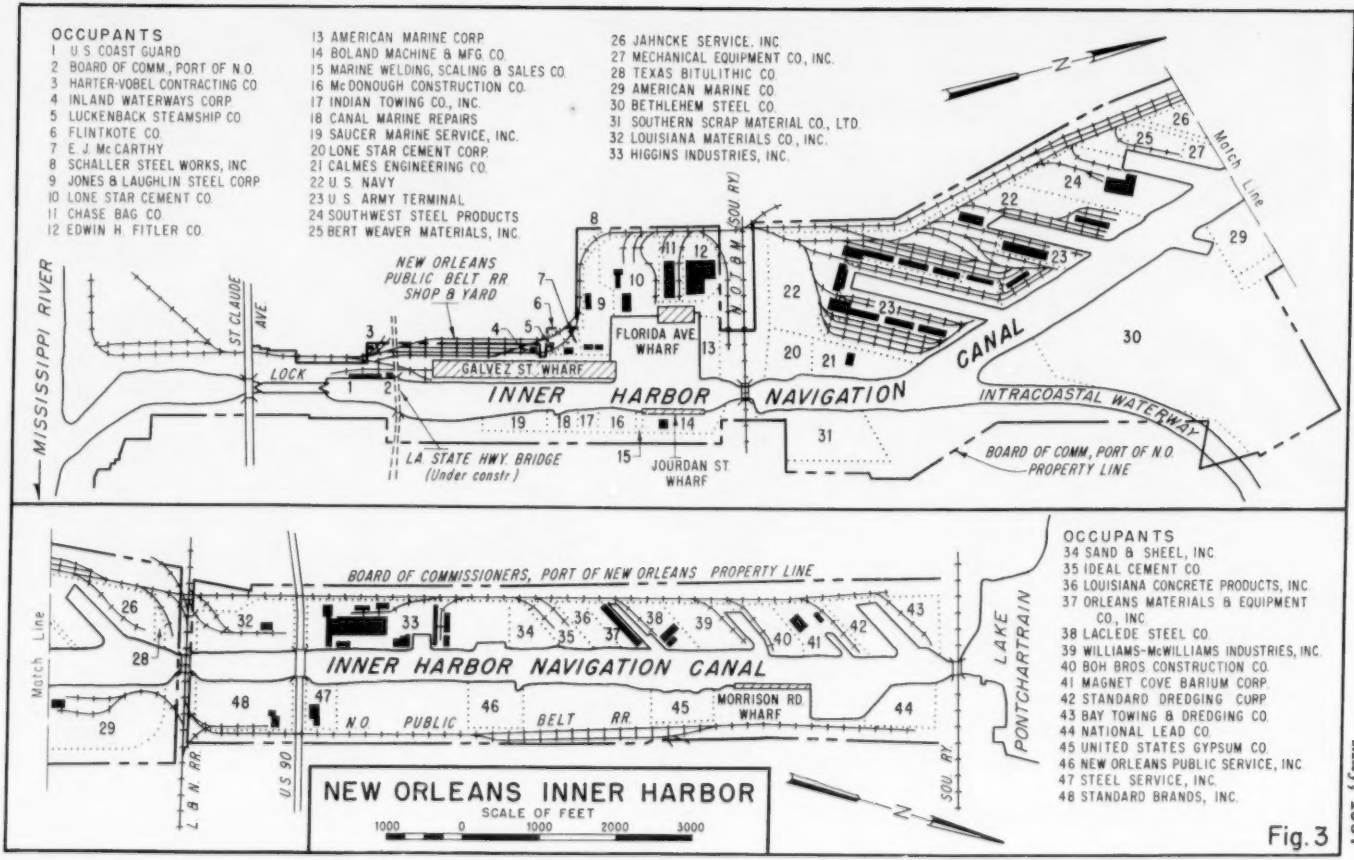

OCCUPANTS
1 U.S. COAST GUARD
2 BOARD OF COMM., PORT OF N.O.
3 HARTER-VOBEL CONTRACTING CO.
4 INLAND WATERWAYS CORP.
5 LUCKENBACK STEAMSHIP CO.
6 FLINTKOTE CO.
7 E. J. McCARTHY
8 SCHALLER STEEL WORKS, INC.
9 JONES & LAUGHLIN STEEL CORP.
10 LONE STAR CEMENT CO.
11 CHASE BAG CO.
12 EDWIN H. FITLER CO.
13 AMERICAN MARINE CORP.
14 BOLAND MACHINE & MFG. CO.
15 MARINE WELDING, SCALING & SALES CO.
16 McDONOUGH CONSTRUCTION CO.
17 INDIAN TOWING CO., INC.
18 CANAL MARINE REPAIRS
19 SAUCER MARINE SERVICE, INC.
20 LONE STAR CEMENT CORP.
21 CALMES ENGINEERING CO.
22 U.S. NAVY
23 U.S. ARMY TERMINAL
24 SOUTHWEST STEEL PRODUCTS
25 BERT WEAVER MATERIALS, INC.
26 JAHNCKE SERVICE, INC.
27 MECHANICAL EQUIPMENT CO., INC.
28 TEXAS BITULITHIC CO.
29 AMERICAN MARINE CO.
30 BETHLEHEM STEEL CO.
31 SOUTHERN SCRAP MATERIAL CO., LTD.
32 LOUISIANA MATERIALS CO., INC.
33 HIGGINS INDUSTRIES, INC.
MISSISSIPPI RIVER
ST CLAUDE AVE
LOCK
NEW ORLEANS PUBLIC BELT RR SHOP & YARD
GALVEZ ST. WHARF
FLORIDA AVE WHARF
L & N R.R.
SOU. RY.
INNER HARBOR NAVIGATION CANAL
INTRACOASTAL WATERWAY
LA. STATE HWY. BRIDGE (Under constr.)
JOURDAN ST WHARF
BOARD OF COMM, PORT OF N.O. PROPERTY LINE
Match Line
BOARD OF COMMISSIONERS, PORT OF NEW ORLEANS PROPERTY LINE
N.O. PUBLIC BELT RR
MORRISON RD. WHARF
LAKE PONTCHARTRAIN
US 90
NEW ORLEANS INNER HARBOR
SCALE OF FEET
1000 0 1000 2000 3000
OCCUPANTS
34 SAND & SHEEL, INC.
35 IDEAL CEMENT CO.
36 LOUISIANA CONCRETE PRODUCTS, INC.
37 ORLEANS MATERIALS & EQUIPMENT CO., INC.
38 LACLEDE STEEL CO.
39 WILLIAMS-McWILLIAMS INDUSTRIES, INC.
40 BOH BROS CONSTRUCTION CO.
41 MAGNET COVE BARIUM CORP.
42 STANDARD DREDGING CORP.
43 BAY TOWING & DREDGING CO.
44 NATIONAL LEAD CO.
45 UNITED STATES GYPSUM CO.
46 NEW ORLEANS PUBLIC SERVICE, INC.
47 STEEL SERVICE, INC.
48 STANDARD BRANDS, INC.

Fig. 3

HYDROGRAPH
INNER HARBOR NAVIGATION CANAL AT NEW ORLEANS
NORTH OR LAKE END GAGE OF LOCK

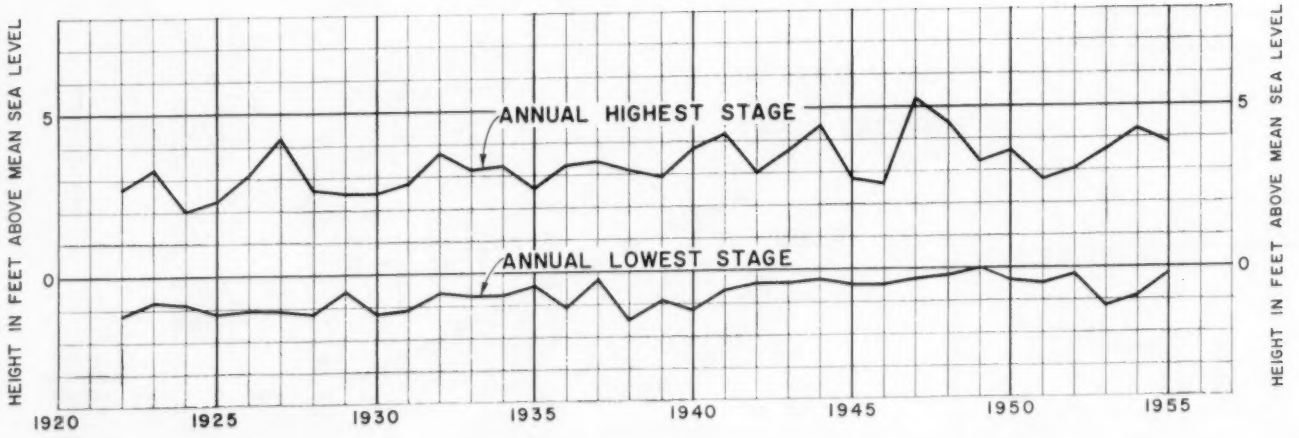
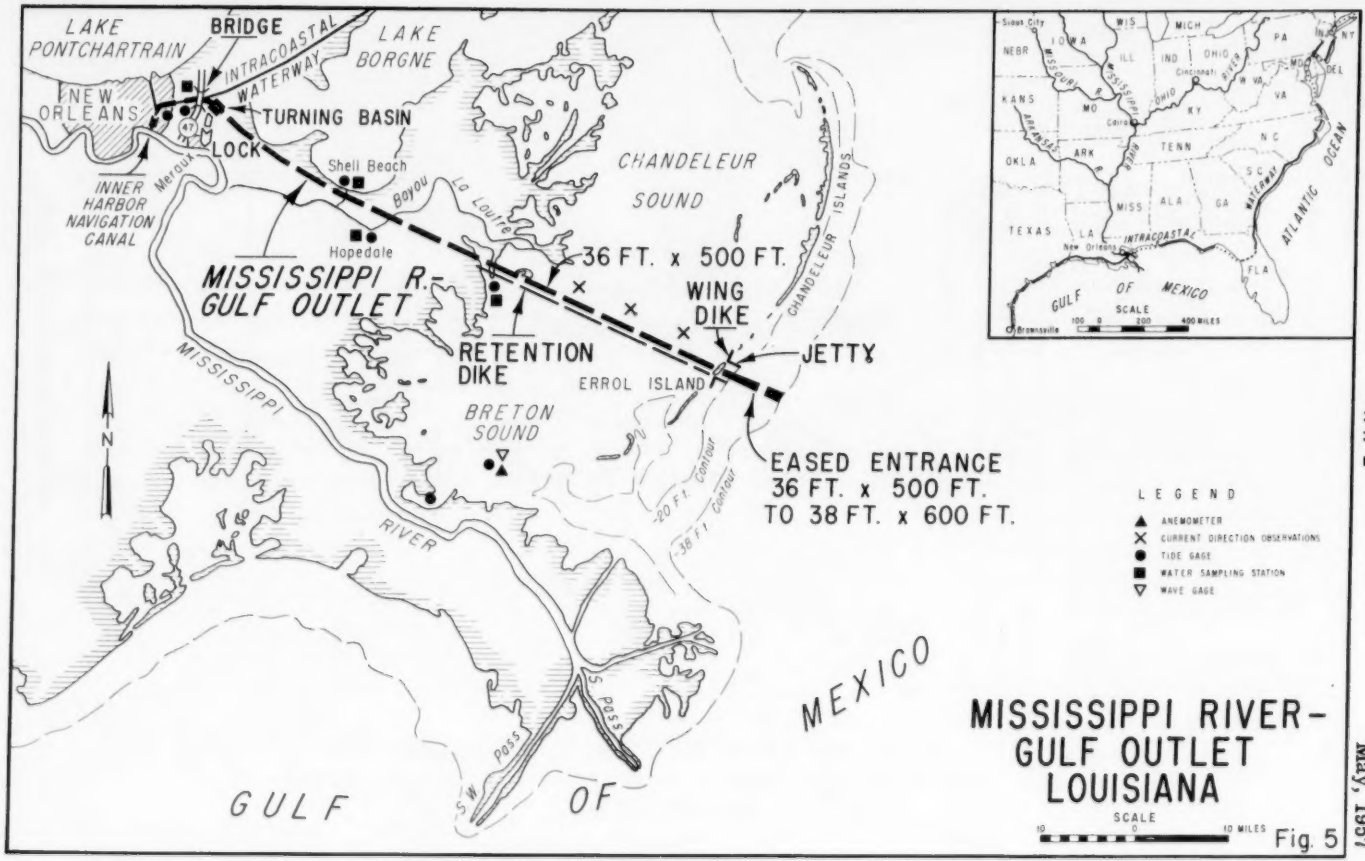
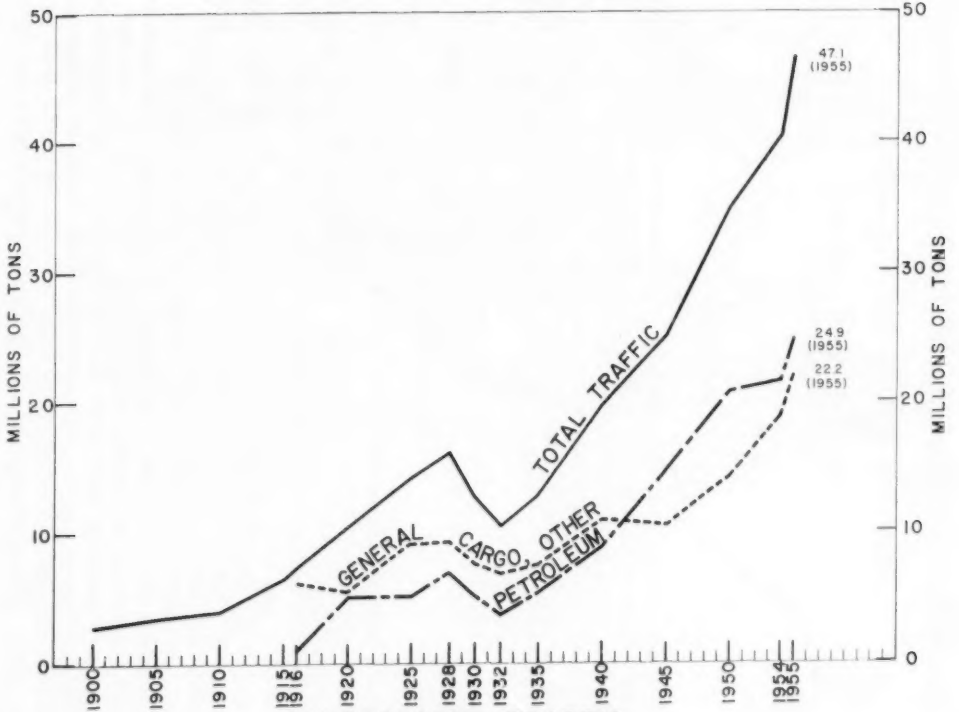


Fig. 4

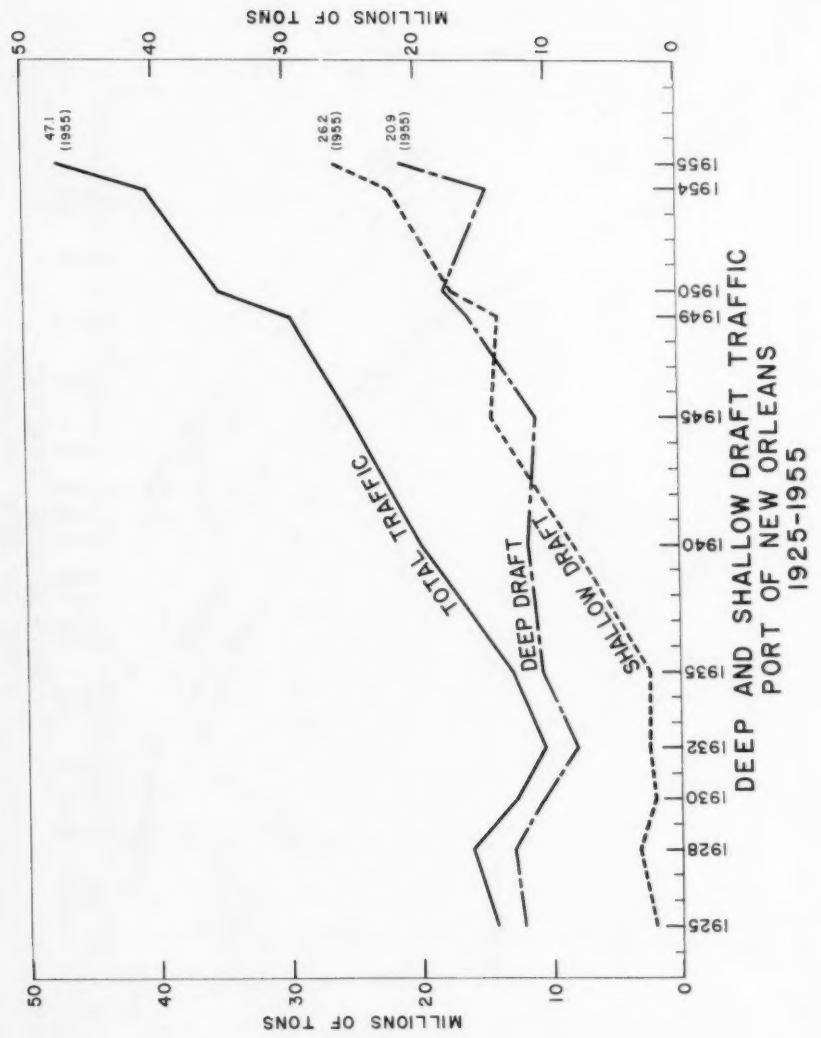

LAKE PONTCHARTRAIN
BRIDGE
INTRACOASTAL WATERWAY
LAKE BORGNE
NEW ORLEANS
TURNING BASIN
LOCK
Shell Beach
Bayou La Loutre
CHANDELEUR SOUND
CHANDELEUR ISLANDS
INNER HARBOR NAVIGATION CANAL
Mereaux
Hopedale
MISSISSIPPI R.-GULF OUTLET
36 FT. x 500 FT.
WING DIKE
JETTY
RETENTION DIKE
ERROL ISLAND
MISSISSIPPI RIVER
BRETON SOUND
-20 Ft. Contour
-36 Ft. Contour
EASED ENTRANCE
36 FT. x 500 FT.
TO 38 FT. x 600 FT.
LEGEND
ANEMOMETER
CURRENT DIRECTION OBSERVATIONS
TIDE GAGE
WATER SAMPLING STATION
WAVE GAGE
SW PASS
S PASS
GULF OF MEXICO
MISSISSIPPI RIVER-
GULF OUTLET
LOUISIANA
SCALE
10 0 10 MILES
Fig. 5
Sioux City
WIS
MICH
NEBR
IOWA
ILL
IND
OHIO
PA
N.Y.
MISSOURI
MISSISSIPPI
OHIO
RIVER
W. VA.
MD.
DEL.
KANS
MO
Cairo
KY
VA
ARKANSAS
ARK
TENN
N.C.
OKLA
S.C.
MISS
ALA
GA
INTRACOASTAL WATERWAY
ATLANTIC OCEAN
TEXAS
LA
New Orleans
FLA
GULF OF MEXICO
SCALE
100 0 200 400 MILES
Brownsville



WATERBORNE TRAFFIC
PORT OF NEW ORLEANS
1900-1955

Fig. 6

Fig. 7


DEEP AND SHALLOW DRAFT TRAFFIC
PORT OF NEW ORLEANS
1925-1955
MILLIONS OF TONS
0
10
20
30
40
50
1925
1928
1930
1932
1935
1940
1945
1949
1950
1954
1955
TOTAL TRAFFIC
DEEP DRAFT
SHALLOW DRAFT
47.1 (1955)
26.2 (1955)
20.9 (1955)

Paper 1241 WW 2

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

MOBILE HARBOR AND SHIP CHANNEL[a]

Harold E. Bisbort,[b] M. ASCE
(Proc. Paper 1241)

SYNOPSIS

The Port of Mobile is located at the head of Mobile Bay, an arm of the Gulf of Mexico having natural depths inadequate for deep-draft vessels. Federal interest in the provision of dredged channels connecting the port with the Gulf began in 1826. This paper describes the gradual improvement of the channel through the ensuing years and traces the history of its effect on commercial and industrial activity of the port and its hinterland.

INTRODUCTION

Mobile Bay is very shallow, having an average natural depth of about 10 feet, and originally a controlling depth over the bar at the mouth of the river of about 5 feet. The pass between the bay and the Gulf is about 3 miles wide and the original natural controlling depth over the entrance bar was about 23 feet. The mean range of tide is about 1.2 feet at the lower end of the bay and 1.5 feet at the upper end. The extreme range, except during storms, is about 3.5 feet. Hurricane winds have raised the water levels in the bay to as much as 11.6 feet above mean low water and depressed it to 9.7 feet below that datum. Both the bay and the entrance bar require dredging to provide adequate depths for ocean-going vessels. Channel improvement by the Federal Government was commenced in 1826 and through the years the dimensions have been increased gradually, commensurate with the increasing dimensions of vessels calling at the port. Essentially, the harbor project now consists of a bar channel 38 feet deep and 600 feet wide, a bay channel 36 feet deep and 400 feet wide and a river channel 36 feet deep over varying widths. Over-all

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a. Presented before Waterways and Harbors Division, ASCE, Jackson, Miss., February 18, 1957.

b. Col. and Dist. Engr., Mobile Dist., Corps of Engrs., U. S. Dept. of the Army, Mobile, Ala.

length of the main channel from the Gulf to the highway bridge over Mobile River is about 40 miles. Depths of 42 feet on the bar and 40 feet in the bay and river channels have been authorized and will be dredged whenever Congress appropriates funds for the purpose.

The geographical limits of the Mobile Engineer District embraces, in addition to Mobile, four other deep-water harbors, Gulfport, Pensacola, Panama City, and Port St. Joe, actively engaged in foreign and coastwise trade. With the exception of Gulfport, these are in sheltered embayments possessing large areas with natural depths adequate for some of the larger freighters afloat and in which siltation problems, requiring expensive maintenance costs, are relatively non-existent. They are important components of our national transportation system, but in spite of their natural advantages they have not kept pace with the phenomenal growth of Mobile, Alabama's only seaport. This Gulf coastal port, for which Uncle Sam has over the years paid out some $20,000,000 to overcome troublesome shoaling problems, now handles nearly four times the tonnage of the other four mentioned ports combined. The story of Mobile's growth and the factors which boosted it to a position of commercial eminence today is an interesting one.

History

The port of Mobile is strategically situated at the head of a virtually landlocked bay and at the mouth of a navigable river system which extends through one of the richest mining and industrial regions in the South. The city was once the site of a peaceful Indian settlement known as "Maubila." "Maubila" was the capital of a large tribe of Choctaw Indians known as Maubilians, which translated into English very appropriately means "canoe paddlers."

Mobile Bay has been known by more than one name. The Spanish first called it "Spiritu Santo" and the English, noting its shape, designated it at one time as "Gunstock Bay." The bay was first discovered and explored in 1519 by the Spanish Admiral de Pineda, who recognized and noted its strategic position and other inherent advantages on the expedition map sent to Spain the following year. The Spaniard Tristan de Luna's expedition of 1559 next visited Mobile Bay, but his efforts to establish a post there met with failure. The Spanish explorers left nothing tangible to be associated with Mobile Bay except nautical charts, and those charts were later to furnish the French explorers with geographical knowledge of the area. Under the leadership of the Canadian seaman Bienville who made use of the Spanish charts, a temporary post was established in 1699 on Dauphin Island opposite the entrance to Mobile Bay. In 1702 a permanent French colony known as Fort Louis de la Louisianne was established by Iberville's younger brother, Bienville, on Mobile River at 27-Mile Bluff, which, as the name implies, was about 27 miles above the present city near the town now known as Mt. Vernon.

Settlement of Fort Louis was simultaneous with the establishment of a permanent post on Dauphin Island. The French supply ships could not sail up Mobile Bay because of the shallow water. At that time, however, an excellent, crescent shaped anchorage was available in a sheltered cove between the eastern end of Dauphin Island and a small sand island to the south. The anchorage, said to be 30 to 35 feet deep and large enough to accommodate from 15 to 30 ships, was connected at its western end with the Gulf by a pass 20 feet

deep and as wide as the length of a French ship. At the little harbor, supplies destined for Fort Louis were transferred to shallow-draft vessels and freighted up the bay and river. A warehouse was built on Dauphin Island in which to store goods awaiting the transfer.

Fort Louis was the first capital of the vast Louisiana territory and Dauphin Island, 60 miles to the south, served as its port for 15 years. The fort at 27 Mile Bluff was abandoned in 1711 and the colonists moved downriver and settled at the present site of Mobile. Before that year, the French had been contemplating the move so as to reduce the distance between the fort and the warehouses on Dauphin Island and to place the settlement within earshot of signal cannon on the island. The immediate cause of the change, however, was a river flood and high tides resulting from a hurricane in August 1711.

Mobile prospered until May 1717, when a severe southwest storm washed sand into the narrow pass connecting the Dauphin Island anchorage with the Gulf, thus effectively bottling up the harbor. Communication by sea during colonial times was essential and the disaster at Port Dauphin forced the French to shift the capital of the Louisiana territory to Biloxi. Mobile was retained merely to serve as a supply depot for Indian trade along the waterways leading into the hinterland.

Historians tell us that it is a geographical rule for every one of the important cities of the world to be built at a crossroads or at the junction of vital trade routes. Upon examination of the map, it may be seen that Mobile is a nerve center, with trade routes radiating from the port of entry northward along the Alabama and Coosa, the Tombigbee, and the Warrior Rivers into a rich and productive land, and along sheltered intracoastal routes to Pensacola, Biloxi, and New Orleans. All these natural physiographical advantages combined to encourage the inevitable growth of the port. Also, the political advantage of being Alabama's only seaport in modern times has placed Mobile in a decidedly better competitive position from the standpoint of ocean shipping than many other Gulf ports. Needless to say, Mobile's setback caused by the deterioration of the Dauphin Island anchorage was relatively shortlived. Shallow-draft vessels could still be accommodated at that location, but it was still necessary for the larger supply ships to anchor in lower Mobile Bay and transfer their cargo to lighters for the trip up the bay.

Control of the new colony passed from the French to the British in 1763, then to the Spanish in 1780. Mobile hoisted the United States flag in 1813, when General Wilkinson siezed the town from Spanish rule during the war of 1812. Mobile was granted a town charter in 1814 and a city charter in 1819 shortly after Alabama was admitted to the Union. What is now Mobile County had a population in 1820 of approximately 2,700 inhabitants.

As is the case now, port facilities in the early years were situated on the west bank of the Mobile River, extending from its mouth for several miles upstream. Obstructive sand bars just below the mouth prevented direct access to the city wharves, but vessels drawing 8 feet or less could bypass the shoals by sailing 12 miles up one of the deeper distributaries in the Mobile delta to a junction with the main stream, thence back down the main stream to Mobile. Vessels of deeper draft still anchored in lower Mobile Bay for interchange of cargo with "lighters" or shallow-draft river boats. The circuity of the route to the port and the added expense of lightering cargo from the lower anchorage were responsible at least in part for the development of a port at Blakely, Alabama, on the comparatively deep Tensaw River across

the Mobile Delta about 10 miles northeast of the present Mobile. Blakely was an official port of entry of the United States, with a customs office, a shipyard, and a population in 1820 of about 3,500 inhabitants.

Channel Improvements

It is axiomatic that the life of a port depends on deep-water access from the sea to the docks, without intermediate lighterage requirements. In 1825 a U. S. Engineer officer was stationed at Mobile and appointed "Superintending Engineer of the construction of Fort Morgan and of the improvement of Mobile Harbor, Pass aux Herons, and the Pascagoula." Federal interest in an adequate harbor at Mobile was inaugurated the following year, May 20, 1826, when Congress appropriated $10,000 to be used for removing obstructions and increasing the channel depths to the port. With this money and later appropriations, Pinto Pass was improved, opened to traffic in 1829, and maintained until the year 1837. Ocean shipping was facilitated by this program but many of the larger ships were still compelled to anchor in the lower bay and transfer cargo to lighters for the trip to Mobile.

Beginning in 1837 and extending to 1857 the Army Engineers dredged and maintained a depth of 10 feet over the obstructive bars at the mouth of Mobile River so that larger vessels would have direct access to the city. This was a period of great prosperity for the port and its trade area. River traffic made up the life of the city and the state. The plantations along the Tombigbee and Alabama River systems furnished an abundance of products, with cotton predominating, for export through Mobile to England, France, Germany, and many other countries. The packet boats which transported the products of inland Alabama to Mobile also brought many wealthy planters who spent their profits in the port city. By 1860 the annual import and export trade grew to some $40,000,000 and the city grew from a population of about 2,000 at the inception of American rule to nearly 30,000 in 1860.

In 1839 Captain Grant, a retired Army Engineer, opened up Grant's Pass to a depth of 6 feet between Mobile Bay and Mississippi Sound as a toll channel to connect Mobile with New Orleans by a protected inside route. This appears to be the first step toward establishing what is now the Gulf Intracoastal Waterway, an important trade route in the commercial life of the city.

The Port of Blakely declined in importance after the improvement of the channels approaching Mobile. However, by 1860, those channels had shoaled to a depth of 7 1/2 feet and during the war between the states the Confederates strengthened the defenses of the city by blocking the approaches through Choctaw and Pinto Passes with multiple rows of piling and obsolete vessels filled with stones. When trade was resumed after the war, merchant vessels were once again forced to take the devious route through one of the delta streams to reach the city docks.

By 1876 the obstructions had been removed and a channel reopened through the bars at the mouth of the river to a depth of 13 feet. Thereafter, the improvement of the ship channel through the bay kept pace with the rapid growth and development of the interior country. There is no way the old question "which came first, the chicken or the egg" can be truthfully answered, but it is borne out by statistical records that each time the harbor channels were deepened, waterborne commerce at the port rose to new heights. A detailed account of how much and specifically for what reasons the channel dimensions

were increased would require many pages. A good example of the way the channel depth affected the economic well-being of the port city may be found by studying the history of the era immediately following the Civil War. During that period only one railroad extended from Mobile into the interior and most of the cotton, which comprised the principal export commodity of all southern ports, was shipped to Mobile by river steamer, then by lighter to the fleet anchorage in lower Mobile Bay. The other ports in the South were well served by rail lines and most had deep water access to their docks. Rapid transit offered by the railroads induced the planters and merchants in the inland points to do business with them, and since the railroads were more interested in ports with deep water at the wharves, Mobile suffered the consequences. Cotton exports declined from over $12,000,000 in 1877 to $3,000,000 in 1882. The rapid decline in cotton shipments affected every line of business—unemployment was widespread and there were less profits for the brokerage firms and others that handled it. Through the concerted efforts of local tradesmen, a channel improvement program was begun in 1870 by the Federal Government and carried on until a continuous waterway was available from the wharves to the Gulf, deep enough to eliminate expensive lighterage operations. History records that the improvement in waterborne commerce which followed was quick and marked; all business firms were given new life, and the depression period was eventually ended.

Briefly, the bay and river channel depths were increased by the Federal Government from 10 to 13 feet during the period 1870-1876, to 17 feet during the period 1880-1889, to 23 feet during the period 1889-1896, to 27 feet from 1910 to 1914, to 30 feet from 1918 to 1926, and to 32 feet during the period 1930-1934. The bar channel first received attention in 1902 when Congress authorized dimensions of 30 by 300 feet. In 1917 the authorized dimensions of the bar channel were increased to 33 by 450 feet and in 1930 to 36 by 450 feet. From time to time various side channels and turning basins were added to the project. In the 1954 River and Harbor Act, Congress authorized dimensions of 42 by 600 feet across the entrance bar, 40 by 400 feet in the bay, and a depth of 40 feet in the river channel as far north as the highway bridge, including a new turning basin opposite Magazine Point. The first phase of this development, consisting of a 38-foot depth over the bar and a 36-foot depth in the bay and river, is now under contract and almost completed. Actually, in order to compensate for shoaling expected to take place in the bay and river channels during the long dredging process, the contractor is being required to excavate to a depth of 38 feet, with one foot allowable overdepth. This first phase of improvement is requiring the removal of nearly 34,000,000 cubic yards of material, and about 20,000,000 cu. yds. additional dredging will be required to provide the 40-foot project. The 40-foot project will be provided when found warranted by the demands of shipping and if Congress appropriates funds for the purpose.

Dredging Problems

In the 130 years beginning in 1826, the United States Government has spent about $8,700,000 for new work dredging in Mobile Bay and River and nearly $11,300,000 for maintenance to restore authorized channel dimensions from time to time. During the period beginning in 1870 and ending in 1956 the quantity of material actually removed during dredging operations aggregated

nearly 320,000,000 cubic yards. In 1870, bay channel dredging was contracted for 50 cents per cubic yard. By 1875 dredging was being accomplished for 17 cents per cubic yard and from that year on, unit costs decreased gradually, as efficiency of excavating machinery increased, to about 7 cents per cubic yard in 1900. In more recent years maintenance costs have usually fluctuated from as low as 3 cents to about 6 cents, with new work costs slightly higher. The present contract price per cubic yard for new work in the bay is 8.27 cents and in the river, 15.47 cents. Excavation in the bar channel, for which a Government-owned hopper dredge is employed, usually costs on the order of 18 cents per cubic yard.

Until the early part of the 20th century, excavation in the bay and river channels was accomplished by dipper, or clam shell dredges, the material being loaded on scows and hauled to the spoil areas. During the early history of the project, the clam shell dredges removed on an average about 2,000 to 4,000 cubic yards per day. Toward the end of the 19th century and during the early 20th century capacities of the dredges reached about 7,000 to 10,000 yards per day. Between about 1900 and 1910 the hydraulic dredges with their tremendous capacities for excavation work began to make their appearance on area projects. The large hydraulic dredges in use on the present contract for enlarging the bay and river channels, working around the clock, remove during an average day amounts ranging from 28,000 to 40,000 cubic yards, depending on the particular section of the channel being dredged and type of material encountered. The largest of the three dredges employed in this work has a 30-inch suction line, 27-inch discharge, and a 2100 horsepower diesel engine on the pump. The Government-owned hopper dredge usually employed on bar channel dredging is equipped with two pumps, each with 1150 horsepower motors, 30-inch suction pipes, and 27-inch discharge lines. The bin capacity is 3,000 cubic yards.

Based on the 22-year record during which the 32-foot project was in effect, the average rate of siltation in the bay and river channels, as determined from maintenance dredging records, amounted to 3,556,000 and 410,000 cubic yards annually, respectively, or a total of nearly 4,000,000 cubic yards. The degree to which the shoaling rate in the bay and river has varied with channel cross-sectional areas is illustrated in the following tabulation:

Min. channel dimensions*	Year completed	Average annual maintenance dredging
23' x 50' to 100'	1896	1,127,000
27' x 200'	1914	3,624,000
30' x 300'	1926	3,661,000
32' x 300'	1934	3,966,000 cu. yds.

*Length of channel about the same in all cases.

Records of maintenance dredging operations and allocation of yardage removed between new work and maintenance for the 23-foot project are not clear and the annual average derived therefor in the above table should not be taken as conclusive. For the remaining projects, the available evidence does not indicate that successive increases in channel dimensions have resulted in particularly significant increases in the annual rate of filling.

Previous investigations and studies of the Mobile Bay channel problem disclosed that shoaling is not largely due to caving of the banks, which appear to be relatively stable on a 1 on 5 slope, nor does the bed of the channel rise or the sides flow in. Apparently, siltation is due principally to the tidal currents, which generally do not coincide with the direction of the dredged channel. The material carried by the cross currents, either in suspension or as bed load, deposits in the channel and due to the reduction in current velocity, is not carried over the opposite bank.

Maintenance operations are performed each year for designated sections of the main channels, the work being programmed so that each section will be redredged in three to four year cycles. The bar channel is subject to filling from littoral material moving predominately from east to west and is redredged about every two or three years, the average shoaling rate, based on a 10-year record for the 36- by 450-foot channel amounting to about 178,000 cubic yards annually.

Material from bay dredging operations is discharged through floating pipe lines to spoil dumps in open water 2,000 feet from the edge of the channel, both on the east and west sides. Through these parallel spoil mounds, openings are left at such locations as may be required by small boat traffic. There is no evidence to indicate that much, if any, material from the spoil areas finds its way back to the channel. Most of the material from river dredging is discharged through shore pipe to spoil areas along the eastern edge of Pinto Island and east of the highway on Blakely Island. Temporary easements have been obtained from property owners for this purpose and efforts are being made to secure either perpetual easements or long-term leases for some areas in order that spoil from future maintenance operations will not present disposal problems.

Growth of Commerce and Industry

Many factors have influenced the growth of commerce and industry at the port of Mobile. The French settlers, who are by nature essentially traders, imported blankets, cloth, and axes from the mother country to trade with the Indians for skins and furs to be exported to France and England. Miscellaneous items such as those made up the bulk of the waterborne commerce at Mobile during early colonial times. Then in 1783 the cotton gin was invented, making the growing and marketing of cotton a profitable venture and of worldwide economic importance. The greater portion of cotton grown in upstate plantations was transported via the Alabama and Tombigbee River systems to Mobile for export to the mills of Europe; the strategic position of Mobile in this trade made it one of the leading ports on the Gulf and a rich economic prize for the European countries which competed for the possession.

The second milestone, the railroad building era after the Civil War, fostered the development of the second major export commodity. The railroads tapped vast forests of virgin pine and hardwoods in the hinterland and the lumbering business which flourished in the ensuing years furnished cargoes to vessels leaving Mobile for many destinations. The railroads, by extending the boundaries of the port's trade area, tended to diversify the import, export, and coastwise trade.

The city of Birmingham was established in 1871 in the midst of a region rich in coal and iron ore deposits and other minerals needed in production of

iron and steel. The steel industry which grew up in that region during the latter part of the 19th century and the improvement of the Warrior-Tombigbee Waterway to provide a cheap transportation route to tidewater further enhanced Mobile's importance to the economic welfare of the State. During the latter 19th century and early 20th century cotton, timber, lumber, coal, iron and steel, sulfur, bananas, sisal grass, and manufactured articles made up the bulk of the export-import trade. Tonnage handled over the city wharves grew steadily from less than half a million in 1880 to three million in 1925.

Many developments heretofore described contributed to the growth of Mobile as a port, but, aside from the deep water channel itself, the construction in 1928 of the Alabama State Docks and Terminals was probably the most significant. In 1948 the Department of Commerce's weekly periodical "Foreign Commerce" in commenting on the State Docks stated that ". . . they form the strategic link between the hinterland and the waterways of the world. They represent the State's successful effort to increase the flow of commerce through the port to the economic benefit of the whole State." From 4,000,000 tons of commerce handled in 1928 to 15,000,000 tons in 1955- that is statistical proof of their beneficial effect on the port.

Principal units of the docks system include: three piers 1,600 feet long and a riverside wharf 1,700 feet long, all with steel and concrete transit sheds and warehouses of modern design providing a total of 2,221,000 square feet of covered storage; a shipside bulk material handling plant with an optimum capacity of 600 tons per hour onto ships and unloading equipment consisting of 5 towers with capacities ranging from 300 to 800 tons per hour each of bauxite, iron ore, manganese, and other bulk materials; a shipside bonded cotton warehouse and high density press; a shipside cold-storage plant with a quick-freezer; a grain elevator with 1,600,000-bushel capacity; and a terminal railroad with a car-holding capacity of 1,000 cars, connecting the docks system and adjacent industries with the four trunk rail lines serving the port. Handling equipment include heavy duty cranes ranging from 30 to 75 tons capacity, small crawler type cranes, numerous warehouse tractors and lift trucks, and several portable conveyors. Additional berthing and terminal facilities to accommodate three ships are now under construction and a building housing an international trade mart is planned. When the new facilities are complete, 28 ship berths will be available at the State Docks and the entire port will afford berthing space for about 55 ships. Total investment in the State Docks is $35,000,000 and a $5,000,000 expansion program is now under way.

Several developments since the State Docks system was placed in operation have combined to promote a further increase in the receipt and shipment of bulk commodities through the port facilities.

The depletion of iron ore reserves in this country, which took a critical turn after World War II, sent the steel companies in search of new sources of supply in South America and other foreign countries. Discoveries of important deposits in Venezuela prompted the inauguration several years ago of shipments to the Birmingham mills through State Docks facilities and a newly constructed bulk handling plant at Choctaw Point at the mouth of the river. The new plant has an unloading capacity of about 1,300 tons per hour; the ore is unloaded by grab bucket onto conveyor belts and then into either rail hopper cars or barges. Ore imports reached 2,000,000 tons in 1955 and within the very near future steel interests expect to import not less than 3,000,000 tons

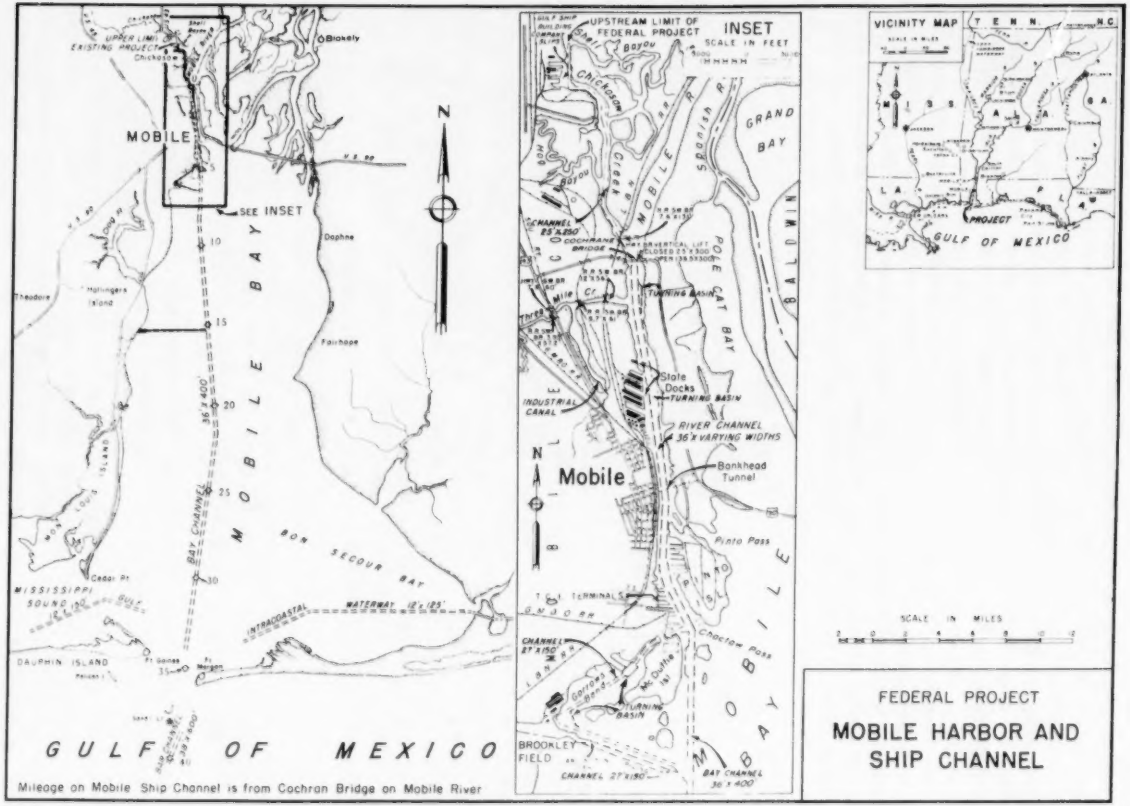
annually. Three of the largest freighters afloat (60,000 tons deadweight) are used in this trade.

Recent discovery of crude oil in Mississippi and Alabama was followed by construction of port outloading facilities, including a storage tank farm just above the State Docks' property, and a crude oil pipe line extending to the Baxterville and Eucutta fields in Mississippi and the Citronelle field in Alabama. Over 1,000,000 tons of crude oil was shipped in 1955 from these facilities by coastwise tanker to Atlantic and Gulf coast refineries, and the volume is growing day by day as new wells are coming into production.

The mounting use of aluminum and aluminum products in the aircraft and other metal industries has increased the demand for bauxite from foreign sources. Nearly 2,500,000 tons of this commodity were imported in 1955 through the bulk materials handling plant at the Alabama State Docks in ships ranging up to 30,000 deadweight tonnage and 32.5 feet in draft.

Future of Port

According to the Bureau of Business Research of the University of Alabama, the port of Mobile is well balanced and exceptionally well equipped, having a fine combination of a good harbor, inland and intracoastal waterway arteries and rail facilities to maintain its position as a leading port. The harbor has terminal and port facilities among the best in the entire country, with all types of ship services available. A canalized waterway via the Mobile, Tombigbee, and Warrior Rivers, affords adequate depths for modern barge traffic between the port of Mobile and the rich industrial Birmingham region in north-central Alabama. Ultimate development of the Alabama-Coosa branch of the Mobile River system, initial development of which is authorized, will further enlarge the system of waterways connecting Mobile with inland industrial sections of Alabama. All indications point to a continued industrial expansion in the Mobile area and further growth of port activity with the new deep 36-foot channel and after the authorized 40-foot channel has been provided.


MOBILE
SEE INSET
Blakely
Daphne
Fairhope
Theodore
Hollingers Island
MOBILE BAY
BON SECOUR BAY
BAY CHANNEL 36' X 400'
INTRACOASTAL WATERWAY 12' X 125'
MISSISSIPPI SOUND
DAUPHIN ISLAND
Cedar Pt.
Ft. Gaines
Ft. Morgan
GULF OF MEXICO
Mileage on Mobile Ship Channel is from Cochran Bridge on Mobile River
INSET
SCALE IN FEET
UPSTREAM LIMIT OF FEDERAL PROJECT
CHICKASAW BAYOU
MOBILE R.
SPANISH R.
GRAND BAY
BALDWIN
CHANNEL 25' X 250'
COCHRANE BRIDGE
TURNING BASIN
POLE CAT BAY
State Docks TURNING BASIN
INDUSTRIAL CANAL
RIVER CHANNEL 36' X VARYING WIDTHS
Mobile
Bankhead Tunnel
Pinto Pass
TERMINALS
Choctaw Pass
MOBILE BAY
CHANNEL 27' X 150'
BROOKLEY FIELD
BAY CHANNEL 36' X 400'
VICINITY MAP
TENN.
N.C.
MISS.
ALA.
GA.
LA.
FLA.
PROJECT
GULF OF MEXICO
SCALE IN MILES
FEDERAL PROJECT
MOBILE HARBOR AND SHIP CHANNEL

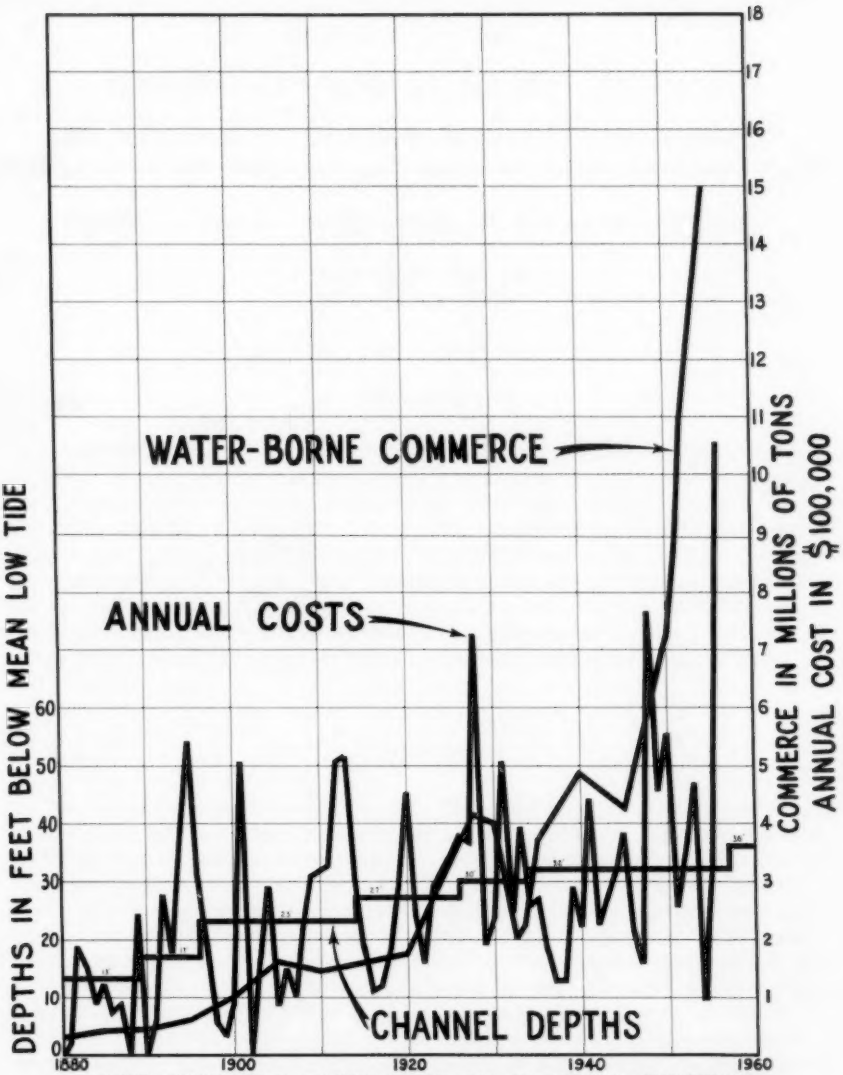


CHART SHOWING INCREASES IN CHANNEL DEPTHS
ANNUAL COSTS, & GROWTH OF COMMERCE IN MOBILE HARBOR

Paper 1242 WW 2

Journal of the
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MISSISSIPPI RIVER - BATON ROUGE TO THE GULF OF MEXICO

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(Proc. Paper 1242)

SYNOPSIS

The lower Mississippi River is navigable by deep-draft shipping for more than 250 miles above its mouth. Commerce moves on this waterway through the ports of Baton Rouge and New Orleans to all parts of the central valley of the United States and all ports of the world. Congress has authorized planning of a tidewater outlet for the port of New Orleans separate from the Mississippi River; however, the river channels which are to be deepened to 40 feet from the Gulf to New Orleans will continue to be used by oil tankers, special cargo carriers and other vessels serving the extensive industrial development along the river between Baton Rouge and the Gulf of Mexico.

INTRODUCTION

The inland port of Baton Rouge, La., 250 miles from the Gulf of Mexico, is at the upper limits of that portion of the Mississippi River regularly navigated by deep-draft, ocean-going vessels. The port of New Orleans, La., on the Mississippi River 115 miles from the Gulf, is a port of world-wide significance and the meeting place of inland water and land transport systems with ocean trade routes to all parts of the world. In 1955 more than 5,200 deep-draft vessels engaged in coastwise and foreign trade visited these ports; 862 of these vessels proceeded up river to Baton Rouge. The port of New Orleans, considered separately, handled more than 47,000,000 tons of cargo which placed it among the leading ports of the nation with respect to tonnage and second to New York in respect to dollar value of commerce. Handling of this tremendous volume of cargo over both inland waterways and ocean routes was made possible by the early development of safe and deep channels through the Passes of the Mississippi River and by progressive improvement and

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systematic maintenance of the river channel at critical locations between the Head of Passes and Baton Rouge.

Since the original settlement at New Orleans the economy of the area has been directly associated with waterborne commerce, beginning in the days of sailing ships through the period of keel boats and river steamboats into the present era of ocean-going super tankers and powerful river towboats pushing integrated fleets of barges. One of the most outstanding periods in the history of the area has been the decade since World War II, during which a tremendous industrial and commercial development has taken place along this stretch of the Mississippi River. Support and extension of this development is dependent on the further improvement of the River channels as well as the construction of the new tidewater outlet and upriver barge channels recently authorized.

Strategic Location and Development

The New Orleans-Baton Rouge Area saw the first permanent settlement by Bienville at New Orleans in 1718. Then, as now, this strategic location at the junction of trade routes was the compelling factor which induced the pioneers to settle in what was then an inhospitable land surrounded by swamps, infested by mosquitoes, alligators and hostile Indians. Protection from floods was necessary from the very beginning and one of the first projects undertaken by Bienville was the construction of levees to protect the fertile lands along the river banks from periodic overflows. Although there are limited areas along this portion of the river where the rich soil is suited for agriculture, commerce and industry have always been the dominant influence and the process of converting large areas of land from agricultural to industrial uses is going on at an accelerated pace at the present time. Along the banks of the river from Baton Rouge to the Gulf are located the nation's largest oil refinery with a daily capacity of 315,000 barrels; the nation's largest aluminum reduction plant with a capacity of 200,000 tons annually, and the second largest alumina plant with capacity for 800,000 tons annually. Among the older industries are the nation's third largest sugar refinery with a capacity of 3.9 million pounds daily and numerous other sugar mills and refineries, the only plant which produces building products from sugar cane fibre (bagasse) and one of only two plants in the United States producing anti-knock compound for motor fuel.

Industrial growth during the past 10 years has been near phenomenal. Plant construction and expansion projects, valued at more than one billion dollars, have been initiated. In the year 1956 new starts have been made on industrial expansion totaling $345,000,000 consisting of 61 new plants and 114 additions to existing plants. The largest single plant built since 1946 is valued at $207,000,000 and at least seven plants are valued at more than $50,000,000 each. The impact of this industrial expansion is felt by the addition of 11,347 new permanent jobs and 18,000 temporary construction jobs to the payroll of this section of Louisiana in 1956, and to support this expansion and other activities in the area the total population has increased 49% from 756,859 in 1940 to 1,130,000 in 1956. The "main stem" of the Mississippi River serves as the supporting structure and has always been the outstanding factor in this development.

Navigation Channels

The first and most formidable problem encountered by the earlier developers was the opening of the Passes of the river to the Gulf of Mexico. Until this job was tackled in earnest by Captain James B. Eads, M. ASCE, in 1875, only feeble attempts had been made to dredge channels across the bars which obstructed the mouths of all the Passes. Unsuccessful efforts had been made as far back as 1726 to deepen the channels by drags and by agitation. The year 1850 may be considered the "year of decision" for it was then that Congress recognized the need for more engineering data and a scientific approach and assigned the Corps of Topographical Engineers the job of making a survey of the alluvial valley of the Mississippi River to devise a plan for protection from floods and to determine the best method for improving the channels at the mouth of the river. This first and very comprehensive survey and report was begun in 1851 and completed in 1861 by Captain A. A. Humphreys, Hon. M. ASCE, who later became Chief of Engineers, and Lieutenant H. L. Abbott, Corps of Topographical Engineers. There followed considerable discussion and many schemes for improving the channels at the mouth of the river. These discussions culminated in the River & Harbor Act of March 3, 1875, in which Congress authorized James B. Eads & Associates to secure and maintain for 20 years a wide and deep channel from the Gulf of Mexico through South Pass to deep water above the Head of Passes. This channel 26 feet deep and 200 feet wide, with central depths of 30 feet, was first obtained on July 8, 1879 and maintained by Captain Eads, with only minor interruptions, until January 28, 1901. On completion of Captain Eads' contract the Corps of Engineers assumed responsibility for maintenance of the South Pass channel. This Pass has continued to improve with only nominal maintenance and there exists today an excellent channel 30 feet deep throughout the length of South Pass and into the Gulf. It was during the period of the Eads' contract that the port of New Orleans underwent a rapid and substantial development and became firmly established as a world port.

The early developers of the river found a natural channel with depths of 40 feet and widths of more than 1,000 feet from above Head of Passes to New Orleans. In order to fully utilize this channel the need for a deeper and wider channel than that in South Pass was recognized and on June 13, 1902 Congress assigned the Corps of Engineers the task of improving Southwest Pass to provide a channel 35 feet deep from the Gulf of Mexico to deep water above Head of Passes. The plan selected for improvement of Southwest Pass followed generally that used by Eads at South Pass and consisted of jetties and contraction works along with dredging to deepen and maintain the navigable channel. The works at the Passes of the Mississippi River have been very ably described in a paper (No. 2648) by W. C. Cobb, M. ASCE, printed in Centennial Transactions, Volume CT-1953, and many technical papers have been written to describe the complex problems associated with this development.

Between Baton Rouge and New Orleans the natural channel in the Mississippi River varies from slightly less than 35 feet to 140 feet in central depths and from approximately 1,300 to 3,000 feet in surface width. Therefore, after the successful improvement of the entrances into the river from the Gulf, the entire lower river below Baton Rouge was opened to world shipping. The channels are now maintained by periodic dredging to remove shoals caused by high water and 230 miles of riverfront from Baton Rouge to Head of Passes are available for deep water terminals and industrial development.

Existing Project

Prior to 1945 several separate projects were authorized for improvement of the river channels below Baton Rouge and on March 2, 1945 Congress combined all previous projects for Improvement of the Passes, the New Orleans Harbor, and the channel to Baton Rouge into one comprehensive project entitled "Mississippi River, Baton Rouge to the Gulf of Mexico." Authorized channel dimensions are as follows: (See Fig. 1)

a) From Baton Rouge to New Orleans: 35 feet deep by 500 feet wide.

b) Within the limits of the port of New Orleans: 35 feet deep with a maximum width of 1,500 feet, measured from a line generally 100 feet from the face of the wharves.

c) From the lower limits of the port of New Orleans to the Head of Passes: 40 feet deep by 1,000 feet wide.

d) Southwest Pass from the Head of Passes to the outer end of the jetties: 40 feet deep by 800 feet wide.

e) Southwest Pass Bar Channel: 40 feet deep by 600 feet wide.

f) South Pass from Head of Passes to the outer ends of the jetties: 30 feet deep by 450 feet wide.

g) South Pass Bar Channel: 30 feet deep by 600 feet wide.

The combined project as now authorized is approximately 80 percent complete.

The major work remaining to be done is the deepening of Southwest Pass from the presently maintained depths of 35 feet to 40 feet as was authorized in 1945. Funds were appropriated in 1956 for engineering studies and preparation of a plan for securing and maintaining the 40-foot channel through Southwest Pass. Field surveys and collection of data are in progress in preparation for model tests to determine the most feasible and economical methods for accomplishing this work.

Additional Improvements Planned

The authorized increase in channel depths from 35 to 40 feet through Southwest Pass can be obtained by dredging; however, improvement of flow conditions is considered necessary to reduce maintenance costs of this deeper channel. Many major ports now maintain 40-foot depths and the economy of using deeper draft vessels for many types of cargo has already been demonstrated. However, this major problem of reducing maintenance to an acceptable cost remains to be solved to open the river for use by super tankers and special cargo carriers now built and building.

Two other projects which will add substantially to the development of water resources in this area have recently been authorized and are now in the planning stage. Congress, by Act of June 30, 1948, authorized the construction of a barge canal, 12 feet by 125 feet, extending from the left bank of the river about 1 1/2 miles above the Mississippi River Bridge at Baton Rouge for a distance of approximately 5 miles along the foot of the high bluffs on which are located many industrial plants. This canal will provide barge terminal

facilities and access to new industrial sites which will extend the Baton Rouge industrial waterfront several miles upstream above the limits of the present head of deep water navigation. This project is authorized as part of the general navigation and flood control project of "Mississippi River and Tributaries."

The other project will have a more far-reaching impact and is one which is expected to be a major factor in the future development of the port of New Orleans. By Act approved March 29, 1956, Congress authorized modification of the project "Mississippi River, Baton Rouge to the Gulf of Mexico" to provide for construction of a seaway canal, known as the Mississippi River-Gulf Outlet, from New Orleans through the marshes and across Chandeleur Sound to the Gulf. This project will provide an alternate channel into the port of New Orleans' industrial area separate and independent from the river channels through the passes.[2]

Engineering Features

Many complex engineering problems have been solved and many remain to be solved in providing the full potential benefits related to deep-draft navigation on the lower Mississippi River. In the development of South Pass, Captain Eads and his chief engineer, Mr. E. L. Corthell, M. ASCE, pioneered in many features of river control engineering. They had studied jetty systems on rivers in Europe and surveys and observations had been made at the passes which were very comprehensive for the particular era. However, it was through a series of trials and errors in execution of their sound fundamental plan that detail answers to their problems were produced and the success of their project assured. The deciding factor in establishing the channels across the bars at the ends of the jetties was the presence of the littoral current flowing generally from east to west across the mouth of the passes, which had not been previously considered by any engineers of the time. By inclining the channel to the eastward instead of dredging in direct prolongation of the jetty channels, this littoral current drifts the sediment-carrying waters to the westward and prevents excessive shoaling from the heavy burden of silt which begins to settle as the fresh river water comes in contact with salt water of the Gulf. The mixing of the salt and fresh water changes the electrical potential of the silt particles which results in coagulation and accelerated settlement of the silt load.

Another important factor is the existence of salt water layers in the Passes underneath the fresh water. This condition was known by Captain Humphreys and mentioned in his report in 1861; however, the full effect of these salt water layers on shoaling and other problems of river development has not yet been fully evaluated. As channels across the bars at the mouth of the Passes were deepened the upstream flow of salt water along the bottom of the channels increased and during extreme low water periods it now reaches above the city of New Orleans. This salt water blocks the lower portion of the channel to outflow of silt-carrying waters and causes shoaling at the zone of contact. These phenomena related to the contact with salt water have been extensively studied and closely related to the shoaling action at the mouths of

2. "Mississippi River—Gulf Outlet," by William H. Lewis and L. C. Brune, Proceedings Paper 1240, ASCE, May, 1957.

the Passes. It will be one of the problem areas receiving further attention through field surveys and model tests in the studies for maintaining the 40-foot project.

Navigation problems further upstream between New Orleans and Baton Rouge are closely related to bank protection, flood control and overall stabilization of the river. Additional works needed below Baton Rouge are being studied at the present time under Congressional authorization for a comprehensive review of the entire project for improvement of the Mississippi River from the Head of Passes to Cape Girardeau, Mo. Under this long range project, caving banks will be protected, channel alignment improved and navigation lanes fixed more consistently. Such work should reduce considerably the maintenance dredging at critical bar crossings above New Orleans.

Maintenance Dredging

Beginning with the earliest attempts at improvement of the river channels, dredging has been necessary to remove sand and silt carried by flood flows and deposited on the bars. Contraction by jetties and other control works has materially aided in reducing the amount of dredging required but the ideal channel and flow conditions which would make dredging unnecessary have not yet been secured in many areas. Shoaling occurs in three general areas as follows:

1. Southwest Pass, South Pass and Head of Passes.
2. New Orleans Harbor.
3. Bar crossings between New Orleans and Baton Rouge.

Frequency and average amounts of dredging at the critical locations are shown in the table on page 7.

During periods of high water the bar channel at the Gulf entrance to Southwest Pass shoals rapidly and it is necessary to dredge continuously to maintain the project depths. This dredging is done by the U. S. Hopper Dredge LANGFITT which is usually able to remove shoals in the bar and lower jetty channels as fast as they form. Experience has shown that at average high water stages the channel shoals at the rate of 50,000 to 80,000 cubic yards per day. The Dredge LANGFITT removes and hauls approximately 50,000 yards per day and additional material is agitated during the dredging process and carried away by the currents, so that under average conditions the dredging compensates for the natural shoaling action in this particular location. The bar channel at South Pass does not shoal so rapidly and is usually cleared by intermittent dredging or restored to full depth after the high water period. During high river stages the increased elevation of the water surface provides ample depths in the channels at the Head of Passes and at bar crossings between Baton Rouge and New Orleans. These deposits are removed during the latter part of the flood period. To accomplish all of this work the Dredge LANGFITT works about five months each year from January to June at the passes and bar crossings and removes approximately 6,500,000 cubic yards of material.

Dredging in New Orleans Harbor following each period of high water is performed by leased dredge of the cutterhead and pipeline type. In an average year approximately 1,000,000 cubic yards of material are removed from shoals within the Federal Project which extends to within 100 feet of the face

MISSISSIPPI RIVER - BATON ROUGE TO THE GULF OF MEXICO

Location	Frequency	Avg. for ea. year dredged (Cubic yards)
Southwest Pass Bar and Lower Jetty Channel	Annually	4,399,808
South Pass Bar and Jetty Channel	7 times in 10 yrs.	239,495
Head of Passes	9 times in 10 yrs.	480,658
New Orleans Harbor: Federal Project	Annually	1,000,000
New Orleans Port Commission	Annually	1,100,000
Philadelphia Crossing (Mi. 182)	2 times in 10 yrs.	422,958
Granada Crossing (Mi. 204)	5 times in 10 yrs.	271,640
Medora Crossing (Mi. 212)	6 times in 10 yrs.	113,237
Red Eye Crossing (Mi. 223)	9 times in 10 yrs.	600,941

of wharves. Material adjacent to the public wharves is removed by the New Orleans Port Commission which dredges approximately 1,100,000 cubic yards annually.

Average annual maintenance cost of the Federal Project for the past five years has been $1,632,833, of which $918,073 has been for dredging.

Wharves and Port Facilities

To facilitate handling of cargo there have been built along the Mississippi River from Baton Rouge to the Gulf modern wharves and handling facilities for all types of cargo. The port of New Orleans now has 96 major docking facilities with public wharves totaling 7 1/2 miles in length. Special wharves for handling bananas, coffee and bulk cargo, a 5,122,000-bushel capacity grain elevator, ship docking and repair facilities and a Foreign Trade Zone for storing and sorting of cargo are also available. Along the river between New Orleans and Baton Rouge there are 19 major docks serving oil refineries and industrial plants and at Baton Rouge there are 17 docks at refineries and industrial plants, two public docks and a public grain elevator. These combined facilities along the lower Mississippi River provide berths for more than 177 ships simultaneously. These deep-draft vessels use the broad natural channels of the river along with the many barge tows and smaller craft without difficulty although controls are necessary at one location during extreme high water stages. In the New Orleans Harbor at Algiers Point are the only known

traffic signal lights for river traffic. Around this "blind corner" the river makes a sharp turn of more than 90 degrees and during high stages current conditions are such that only one ship at a time can safely navigate around the point. The traffic lights are located on the river banks above and below the point and are operated by licensed river pilots who hold ships at safe distances until the channel is clear.

Commerce

Cargo handled through the port of New Orleans in 1955 totaled 47,089,000 tons and the port of Baton Rouge handled 16,489,000 tons. This total of 63,572,000 tons is 42.2% greater than the 44,718,000 tons handled in 1950 and is more than three times the traffic of 1930. Additional traffic on the river, but which does not pass through any local port, brings the total tonnage on the section of the Mississippi River from Baton Rouge to the Gulf to the imposing total of 70,253,000 tons, which is 77% of the total commerce on the Mississippi River between Minneapolis and Head of Passes. In 1896, which was the year the New Orleans Port Commission was organized and prior to development of the navigable channel in Southwest Pass, the total traffic through the port of New Orleans was 2,140,396 tons. Figures 2 and 3 indicate graphically this growth in recent years.

One of the major factors in traffic increase has been the extensive development of oil and gas resources in the last ten years. This development covers most every section of the State of Louisiana but a large part of the most recent development in production, as well as in the refining of petroleum products, is in the southern part of the State and moves over the waterways of South Louisiana. Petroleum and its products constituted 57% of the total tonnage through the combined ports of Baton Rouge and New Orleans in 1955. Pipe lines are moving an increasing amount of oil; however, the coastwise shipments of oil and its products in deep-draft vessels increased more than 365% between 1930 and 1955 and now totals 14,864,000 tons annually. This growth has been given new impetus by exploration and production in the Gulf of Mexico where wells are now being drilled in water depths of 120 feet, and locations 50 miles off shore. This development has increased very materially the number of deep-draft vessels using the river channels. All indications are that the activity will continue to increase in the foreseeable future.

Current Developments

Located at the final outlet of the major watershed of the United States, this area is particularly sensitive to the water resources policies in effect and under consideration by both Federal and local governments. Recent extreme low water conditions on the Mississippi River and tributaries have highlighted many problems relating to maintenance of navigation channels in the upper rivers as well as affecting the supply of fresh water available for users along the lower river. It is significant, however, that navigable depths in this section are not affected by conditions of low flow in the upper river. Due to the proximity with the Gulf the river to Baton Rouge becomes a tidal estuary at low stages and project depths of channels are all referred to Gulf datum. The most adverse effect is that the lack of adequate fresh water flow allows salt water to travel upstream more than the usual distances and affects water

supply for the city of New Orleans and industrial users who depend on the river for their fresh water. Many industries such as chemical, petrochemical, aluminum reduction plants and steam power plants which require large quantities of fresh water either in manufacturing processes or for cooling and other collateral uses have been attracted to the lower river during the war and post-war periods.

Figure 4 indicates the extent of this industrial development which has been especially signigicant since 1939, both in number of manufacturing establishments and in size of plants as measured by number of persons employed.

This current development is related very closely to the water resources of the area and is made possible by more than 25 years of successful flood control on the river and the uninterrupted free navigation to riverside industrial sites. Studies now being made of the entire Mississippi River project have a three-fold objective of:

1. Greater safety from floods;
2. Stability of levee lines and river banks; and
3. Improved navigation channels and current conditions.

It has been established that as the natural forces are curbed and brought under control all improvements along the river tend to become more stable; however, new problems arise with each new development and "the giant" offers a continuing challenge to those concerned with directing its energies and resources for the benefit of the people it serves. The challenge is no less in 1957 than it was when DeSoto first discovered the river in 1541.

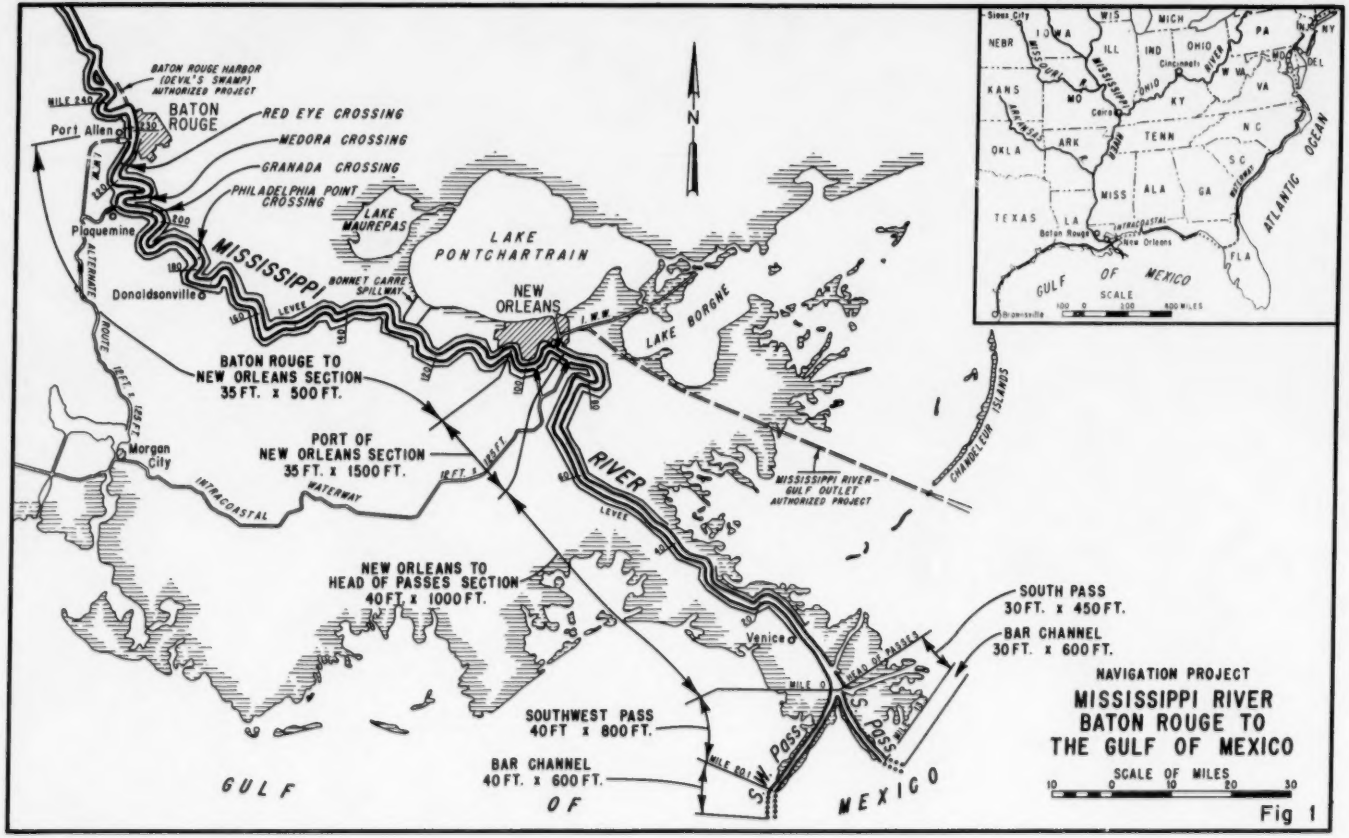
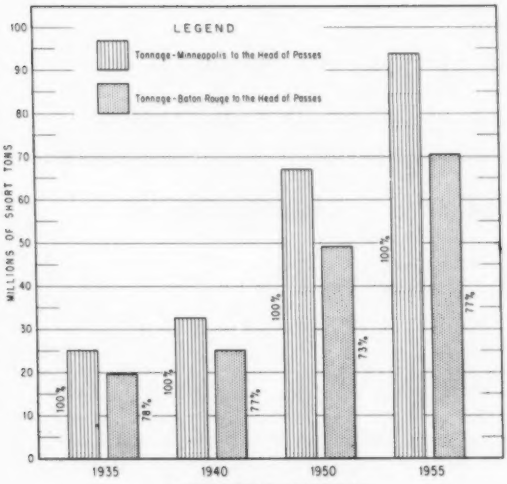
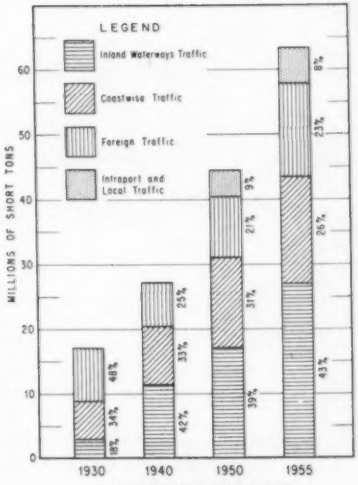

BATON ROUGE HARBOR (DEVIL'S SWAMP) AUTHORIZED PROJECT
BATON ROUGE
MILE 240
Port Allen
MILE 230
RED EYE CROSSING
MEDORA CROSSING
GRANADA CROSSING
PHILADELPHIA POINT CROSSING
LAKE MAUREPAS
LAKE PONTCHARTRAIN
NEW ORLEANS
Plaquemine
MISSISSIPPI
BONNET CARRE SPILLWAY
LEVEE
Donaldsonville
ALTERNATE ROUTE
IFRT
IWW
LAKE BORGNE
BATON ROUGE TO NEW ORLEANS SECTION 35 FT. x 500 FT.
PORT OF NEW ORLEANS SECTION 35 FT. x 1500 FT.
Morgan City
INTRACOASTAL WATERWAY
CHANDELEUR ISLANDS
MISSISSIPPI RIVER-GULF OUTLET AUTHORIZED PROJECT
RIVER
NEW ORLEANS TO HEAD OF PASSES SECTION 40 FT. x 1000 FT.
Venice
SOUTH PASS 30 FT. x 450 FT.
BAR CHANNEL 30 FT. x 600 FT.
HEAD OF PASSES
MILE 0
S. PASS
SOUTHWEST PASS 40 FT x 800 FT.
BAR CHANNEL 40 FT. x 600 FT.
MILE 601
SW. PASS
GULF OF MEXICO
N
Sioux City
IOWA
NEBR
WIS
MICH
ILL
IND
OHIO
Cincinnati
PA
NY
DEL
KANS
MO
KY
W VA
VA
Cairo
OKLA
ARK
TENN
NC
SC
TEXAS
LA
MISS
ALA
GA
FLA
Baton Rouge
New Orleans
ATLANTIC OCEAN
GULF OF MEXICO
SCALE
100 0 100 200 MILES
NAVIGATION PROJECT
MISSISSIPPI RIVER
BATON ROUGE TO
THE GULF OF MEXICO
SCALE OF MILES
10 0 10 20 30

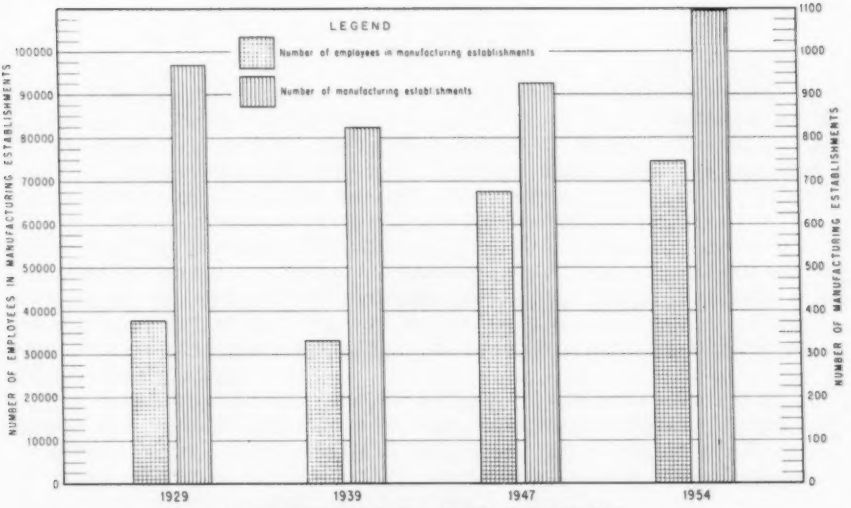
Fig 1



WATERBORNE COMMERCE
MISSISSIPPI RIVER
Fig 2



WATERBORNE COMMERCE
PORTS OF BATON ROUGE AND NEW ORLEANS
Fig 3



MANUFACTURING ESTABLISHMENTS ALONG THE
MISSISSIPPI RIVER-BATON ROUGE TO HEAD OF PASSES

Fig 4

Journal of the
WATERWAYS AND HARBORS DIVISION
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FLOOD CONTROL FOR THE MISSISSIPPI-YAZOO DELTA

C. K. Little[1]
(Proc. Paper 1243)

SYNOPSIS

The control of floods in the Mississippi-Yazoo Delta presents a complex problem because of the multiplicity of sources of flooding and the magnitude and character of the area involved. This paper presents a brief discussion of the problem and of works constructed and planned for its solution.

Description

The Mississippi-Yazoo Delta, known locally as the "Mississippi Delta," comprises nearly four and one-quarter million acres of the alluvial valley of the Mississippi River in the northwest portion of the State of Mississippi. It is an oval-shaped area with apexes at Vicksburg, Mississippi, and at a point on the Mississippi-Tennessee line 190 miles north of Vicksburg. At its widest point, in the latitude of Greenville and Greenwood, the area is about 65 miles in width. The East Bank Mississippi River Levee forms the western boundary and to the east the area is bounded by bluffs, in some places almost vertical, rising 100 to 200 feet above the Delta area. The general slope is to the south at a rate of about one-half foot per mile, and also toward the long axis, both from the Mississippi River and from the eastern bluffs, at a less uniform rate of slope.

The Yazoo River System provides drainage for the Delta area and for an almost equal area of hill lands to the east. The Yazoo, Tallahatchie and Coldwater Rivers generally parallel the hill line and discharge into the Mississippi River at Vicksburg. They form the main stem of the drainage system and, within the limits of their channel capacities, carry the runoff from the hills and the extreme northern portion of the Delta area. Most

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interior drainage reaches the Yazoo River through Sunflower River and Steele Bayou. A general map of the area is shown in Figure 1.

Soils in the area are very fertile and when protected from flooding and adequately drained, high yields of cotton, beans, corn, pasture, rice and other crops are produced. Rainfall is abundant, the average being 51 inches annually, occurring mainly during the months of January through June.

Industry is confined to the larger towns and consists principally of processing agricultural and timber products. Total population was approximately 435,000 in 1950, with eleven towns having populations of over 2,000; the largest being Greenville, with 30,000 inhabitants.

The Flood Problem

The history of the Mississippi-Yazoo Delta has been one of continual struggle against flooding. Prior to the construction of levees, the entire area was subject to flooding from the Mississippi River. Intermittent levees on the higher ground along the Mississippi River were first constructed early in the 19th Century, and a continuous levee from the hills near the Tennessee line to a point some 15 miles north of Vicksburg was completed about 1875. Early levees were inadequate in height and size and disastrous breaks occurred at relatively frequent intervals, the latest being in 1927. After adoption of the Flood Control Act of 1928 by the Congress, levees along the Mississippi River were materially increased in height and cross section until at present the probability of their failure is remote.

Although enlargement of the Mississippi River Levee removed the principal threat of flooding in the area, there remained other sources of damaging floods. Rapid runoff from the hills supplemented by slower runoff from the Delta area resulted in periodic flooding of some 1,630,000 acres along the Yazoo, Tallahatchie and Coldwater Rivers. Portions of the area were sometimes flooded twice yearly, and severe floods occurred in 1932, 1933, 1935, 1937, 1938 and 1939. Flooding became less frequent after 1940, as construction of the flood control features progressed. Because of inadequate channel capacities, some 625,000 acres were subject to less severe but costly damage along the Sunflower River, Steele Bayou and their tributaries; and in addition, about 900,000 acres in the southern part of the area were subject to flooding by backwater from the Mississippi River. This paper deals with plans for the control of flooding in the area after completion of the Mississippi River Levee and other improvements on the Mississippi River. Areas subject to flooding after completion of the Mississippi River Levee are shown on Figure 1.

Flood Protection for the Area

Prior to flooding of the area by the Mississippi River in 1927, no less than 121 local Levee and Drainage Districts were formed in an attempt to provide local flood protection. Works of the local interests were not coordinated and were disconnected, and although improvements in local drainage resulted, they proved entirely ineffective in controlling the larger floods.

Surveys and studies by the Corps of Engineers were authorized in the Flood Control Act of 1928, and beginning with the Flood Control Act approved in June of 1936, flooding of the area was recognized by the Congress as a national problem. The 1936 Act authorized flood protection for that portion of

the area along the Yazoo, Tallahatchie and Coldwater Rivers. Subsequent Acts approved in 1941 and 1944 authorized protection from Mississippi River backwater and the improvement of Sunflower River, Steele Bayou and their tributaries. The language of the authorizing Acts was intentionally broad and permitted the Chief of Engineers wide latitude in determining the most effective and economical means of providing flood protection and major drainage improvements.

Plans for Flood Protection

Studies based on surveys and stream measurements initiated after passage of the 1928-Flood Control Act, indicated that all known means of flood control would be required for protection of lands along the Yazoo, Tallahatchie and Coldwater Rivers. Discharges from each of the three largest hill streams, the Coldwater, Little Tallahatchie, and Yalobusha Rivers, ranged upward to peaks near 100,000 cubic feet per second and a fourth, the Yocona River, contributed peak discharges of about 40,000 cubic feet per second to the Delta streams. In addition, discharges from numerous lesser hill streams and from the Delta area itself, must drain through the Yazoo River system. To carry these discharges, channel capacities were grossly inadequate and ranged from an average of about 20,000 cubic feet per second for the Yazoo River to as little as 2,500 cubic feet per second on Coldwater River at the hill line.

There are few desirable dam sites in the Yazoo River Basin. Stream valleys in the hills are relatively wide and soils are sedimentary in origin. Relatively long dams of earth construction are required. After thorough investigations, sites were selected on the four principal tributaries and reservoirs were designed to control maximum storm series, except in the case of Coldwater River where the maximum storage available at the site was utilized. As finally designed, a total of 3,826,000 acre-feet of flood control storage is available in the four reservoirs; namely, Arkabutla on the Coldwater River, Sardis on the Little Tallahatchie River, Grenada on the Yalobusha River and Enid on the Yocona River. With runoff from 65% of the hill area controlled by reservoirs, protection of the area by means of channel improvements, levees and diversions became practicable.

Design flows from uncontrolled areas below the reservoirs ranged from 15,000 cubic feet per second on the Coldwater River near the hill line to 51,000 cubic feet per second on the Yazoo River at Yazoo City. Enlargement of stream channels to the extent required to carry the design flows was determined to be impracticable from a purely engineering standpoint and to be more expensive than a combination of levees and less extensive channel improvements. Accordingly, channel improvements were designed primarily to carry emptying flows from the reservoirs and to afford sufficient capacity to carry runoff from normal rainfall occurring during the crop-growing season. Channel improvement works consisted of channel clearing, cut-offs where feasible, and where required to provide capacity for emptying flows, stream channels were enlarged. After the channel design had been established, levee grades were determined at the elevation required to contain the design flows.

Because of the necessity of providing for interior drainage to lands protected by the levees, and to reduce backwater on tributary streams, it was

necessary in some reaches to limit the elevations of the design water surface. Limiting the elevation of the design water surface will be accomplished, where necessary, by providing auxiliary channels and floodways. One such auxiliary channel and floodway, 32 miles long and now under construction, will provide a lowering of 7 feet in the design water surface at its head below the water surface which would have obtained without the floodway. The lowering will extend over a reach of approximately 135 miles of the Yazoo River. At locations where other means for caring for interior drainage are impracticable, such as at the towns of Greenwood and Yazoo City, pumping plants are provided.

In addition to the four reservoirs, present plans for protection of Delta lands along the Yazoo, Tallahatchie and Coldwater Rivers provide for approximately 734 miles of improved stream channels, 550 miles of levees and one or two auxiliary channels. A more detailed discussion of design and the effect of the reservoirs and improvement work is contained in an article by George A. Morris, published in "Civil Engineer" in May, 1942.

Yet another phase of the problem is the control of some twelve of the smaller hill tributaries of the Yazoo, Tallahatchie and Coldwater Rivers in the area between the hill line and the main river system. Although their drainage areas average less than 100 square miles each, runoff is high and their slopes are steep. Peak discharges of from 20,000 to 30,000 cubic feet per second at the hill line can be anticipated. Soils in the watersheds and stream valleys are easily eroded and large quantities of silt and sand are transported into the area between the main river system and the hills. Preliminary plans provide for the construction of floodways and silting basins; however, these plans would provide only temporary control. Large reservoirs in the hills appear uneconomical. Studies are now under way in cooperation with the Soil Conservation Service of the Department of Agriculture to determine the feasibility of soil erosion control measures and small flood detention dams, in conjunction with floodways in the Delta Area.

The improvement of Sunflower River, Steele Bayou and their tributaries involves approximately 670 miles of stream channels in the Delta Area. As on the Yazoo, Tallahatchie and Coldwater Rivers, improvements consist of clearing tree growth, the excavation of cut-offs and channel enlargement where required. Channel capacities, ranging from as little as 200 cubic feet per second near the head of the smaller streams to 24,000 cubic feet per second on Sunflower River near its mouth, are designed to provide good agricultural drainage for the major drainage outlets. Lateral drainage channels, where not already in place, will be provided by the local Drainage Districts and other local interests. Low water weirs are being constructed, where feasible, to prevent tree growth by ponding water in the channels.

Present plans for protection from backwater from the Mississippi River provide for a levee 53 miles long, joining the Mississippi River Levee with the one along the west bank of the Yazoo River, and levees around certain areas east of the Yazoo River below Yazoo City. In accord with authorizing legislation, the levees will be constructed at a lower grade than that of the main line Mississippi River Levees, to provide flood storage during extreme floods on the Mississippi River. Floods on the Mississippi River which will overtop the backwater levees are rare.

The Backwater Levees will cross a number of streams, including Sunflower River and Steele Bayou. Large floodgates and pumping plants will be required. The preliminary design provides for about 187,000 acre-feet of

sump storage and an average volume of about 1,000,000 acre-feet to be pumped annually. The total pumping capacity will exceed 15,000 cubic feet per second. However, the plan is being reviewed in connection with preparation of final plans and specifications and values given herein will doubtless be revised. The design is based on an analysis of past floods on the Mississippi, Yazoo and Sunflower Rivers, adjusted to reflect changed conditions. It is an interesting and complex problem in itself, also involving a determination of proper distribution of pumping capacity and flow between Sunflower River and Steele Bayou, and the economic balance between pumping capacities and areas in sump storage. The works planned for protection of the Mississippi-Yazoo Delta are shown on Figure 2.

Benefits and Costs

Although the entire Delta Area comprises some 4,225,000 acres, a large portion of the area is completely protected from flooding by the Mississippi River Levee and not all of the remainder is affected by the works discussed herein. It is estimated that when all presently-planned works are completed, approximately 2,500,000 acres, in addition to those completely protected by the Mississippi River Levee alone, will be benefited. Of the 2,500,000 acres benefited, about 1,600,000 acres will be fully protected from flooding. Based on records of past flood damages and estimates of increased productivity of lands, the ratio of annual benefits to annual costs is estimated to be 2.8 to 1.

The entire project, exclusive of the Mississippi River Levee, is estimated to cost approximately $183,000,000 and was 46% complete as of 1 July 1956. The rate of construction under way for the past 20 years has been largely dependent on the amount of appropriations by the Congress. The four reservoirs which cost approximately $70,000,000 were completed in 1953 and have provided substantial benefits with the completion of each unit. Runoff from a storm in the spring of 1955, which produced as much as 11 inches of rain over a portion of the hill area, was largely retained in the reservoirs. Stages on the Yazoo River at Greenwood were reduced about 9 feet below that which would have obtained without the reservoirs and other improvement works. Channel improvements, now essentially complete along the Yazoo, Tallahatchie and Coldwater Rivers and about 15% complete on Sunflower River and Steele Bayou, have also provided their share of benefits.

Construction of levees and auxiliary channels along the Yazoo, Tallahatchie and Coldwater Rivers is now under way and orderly phasing of future construction would indicate initiation of the Backwater Levees within the next two or three years. Protection from the hill tributaries for the area between the Yazoo, Tallahatchie and Coldwater Rivers and the hills, should follow the construction of levees and auxiliary channels along the main river system.

AREA SUBJECT TO FLOODING IN THE MISSISSIPPI-YAZOO DELTA

SCALE OF MILES
10 0 10 20 30

FIG. 1

FLOOD CONTROL WORKS FOR PROTECTION OF THE MISSISSIPPI-YAZOO DELTA

SCALE OF MILES
10 0 10 20 30

FIG. 2

Paper 1248 WW 2

Journal of the
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Proceedings of the American Society of Civil Engineers

SHORESIDE FACILITIES FOR SPECIAL PURPOSE SHIPS[a]

Howard J. Marsden[1]
(Proc. Paper 1248)

SYNOPSIS

A brief history and the current situation regarding special purpose vessel operations and proposed ship construction program, together with a discussion of the layout, general design criteria, and technical analyses covering roll-on roll-off, lift-on lift-off, container, and conveyor terminals for trailerships, trainships, and containerships. Includes considerations involving berth and pier facilities, transfer bridges, marshalling areas, receiving and delivery areas, and subsidiary facilities areas.

The rising cost of conventional cargo handling methods at America's ports has created great interest in roll-on roll-off, lift-on lift-off and similar types of shipping. These types of operations employ the principle of the transfer between land and sea carriers of loaded highway trailers, rail cars and containers; a version of the unit load principle as against the handling of many individual, small pieces of cargo. The savings in vessel port time and cargo handling costs through the use of these specialized operations are potentially enormous.

Historically, ferries were the first of the roll-on roll-off vessels, and still perform their valuable service daily, plying the waters of many of this nation's harbors. In 1860 trainship service was inaugurated in Scotland to convey rail cars across the Firth of Forth and the River Tay. Subsequently, trainship operations were established in other countries and, in 1892, the first train ferry in the United States began operations on Lake Michigan. With the development and accelerated use of automobiles and trucks, ferries

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a. Prepared for presentation at American Society of Civil Engineers, 1956 Annual Convention, Pittsburgh, Pa., Monday, October 15, 1956.

1. Chief, Div. of Port Development, Maritime Administration, Federal Maritime Board, U. S. Dept. of Commerce, Washington, D. C.

were established to transport those vehicles. Bridges and tunnels, however, have largely replaced ferries on the shorter water crossings but ferries are being increasingly used on long runs. Containers have also been used for many years to transport goods in general cargo ships, and consideration has been given recently to the use of ships specially designed or modified for the efficient handling and transportation of containers. The use of roll-on roll-off and lift-on lift-off methods of handling was accelerated during both World Wars and is continuing in the interest of national defense.

While there are a number of commercial roll-on roll-off and lift-on lift-off deep water vessel operations currently being conducted from United States ports including trailer ships and trainships, the first specially designed ocean vessel for highway trailer transportation is yet to be completed. This vessel is being constructed for the Military Sea Transportation Service and will be able to transport, for example, 305-2 1/2 ton trucks. The only current deep water roll-on roll-off trailership operation is a service between Jacksonville, Florida, San Juan, P. R., and the Virgin Islands. This service uses LST's capable of carrying 52 trailers on two decks which vessels have their propelling machinery removed and are towed. However, a Landing Ship-Dock is currently being converted for this service.

An atmosphere of caution has recently supplanted the enthusiasm prevalent several months ago among prospective builders of such special purpose ocean-going vessels. As long ago as one year, applications for Federal Ship Mortgage Insurance for such vessels were approved in principle by the Maritime Administration. Since then, applications for 20 trailerships, 11 trainships, and 2 train-trailer ships have been received. Many have been approved in principle for Government insurance purposes. However, construction contracts have not as yet been let for any of these vessels. It is apparent that ship operators are taking a second, more technical, look at the over-all situation, particularly the aspects of roll-on roll-off versus lift-on lift-off, as well as the question of loading the complete trailer aboard versus loading only the trailer body; in other words, the question of loading a piece of mobile transportation equipment aboard as against a pure container operation. Already one potential operator has changed his plans from a trailership concept to one envisioning the placement on board of the trailer body only in the form of a cargo container—thus the projected trailership has become a containership.

The vessels currently projected that I have indicated above, plus two government designs, vary widely as to loading media. Some are end loaders, some side-port loaders and some combine both features. One lift-on lift-off type vessel loads and discharges rail cars through a "well" opening in the ship. Other than two such self-loading trainships only three of the above vessels have ship-furnished ramps and even these will need the aid of terminal ramps or transfer bridges at certain ports and under certain tidal conditions.

Because of the wide-spread interest in these special purpose vessel operations the Maritime Administration has been conducting a study of the design criteria for the shoreside facilities necessary for the various types of vessels and the loading and unloading media contemplated. While the current uncertain direction of the trend in this field may make this discussion of terminal design criteria somewhat premature it is considered the following preliminary data will be of interest to the engineer as well as the potential terminal and ship operators.

Generally, the layout of a terminal for such vessels is influenced to a large

extent by the variation of the water level in the berth. The dimensions of the vessel, the frequency of vessel arrivals at a terminal, and the number and characteristics of the cargo units discharged and loaded per berthing determine the type and size of the facilities to be provided.

Now, a word about the cargo for these terminals—the "rolling containers." The size and weight of highway trailers are controlled by state motor vehicle laws. Because of the low density of most general cargo commodities, the amount of cargo carried by a highway semi-trailer rarely exceeds 20 tons. A standard highway tractor has sufficient tractive effort to haul a trailer carrying a maximum load of 25 tons on a 12 percent grade. With a minimum underclearance of 10 inches for the trailer landing gear in the fully retracted position, the 50-foot tractor-trailer combination recommended for trailership operation can negotiate a change in grade of 16% without a vertical curve. With a minimum underclearance of 9 1/2 inches, railroad cars up to 60 feet long can negotiate a change in grade of 6.5 percent without a vertical curve.

Both the trailership and the trainship terminal should have the following principal facility units:

1. The ship berth.
2. Marshalling area—which provides space for the accumulation of rail cars or trailers arranged in the order in which they are to be loaded on the ship and for the immediate storage of units unloaded from the ship. This facility should be as near as possible to the ship berth in order to minimize the number of hauling units needed to unload or load the vessel.
3. Receiving and Delivery Area—in the case of a trainship terminal this is a railroad yard with a capacity of one shipload. In the case of a trailership terminal this is a gate house where trucks are checked and includes some space for trucks awaiting gate clearance, as well as rail tracks and unloading ramps where trailers are handled on flat cars ("piggy-back").
4. Subsidiary Facilities Area—these need not be located at the terminal.
    a. Consolidation and Transfer Area—for the consolidation of outbound LTL (less than truckload) or LCL (less than carload) cargoes and the breakdown of inbound full loads.
    b. Holding Area—for short term storage of cargo units.
    c. Repair and Maintenance Area—for minor repairs to trailers.

The ships berth should be readily accessible to deep water and, if possible, should be so oriented as to permit docking and undocking of ships without the aid of tugs. The choice of the type of berth to be used at a particular location is influenced by conditions of tide, wind, current and accessibility.

Roll-On Roll-Off Terminals

Ship berthing facilities may be grouped into five general types—marginal wharf, finger pier, slip wharf, ferry slip and off-shore berth. Slip wharves and finger piers are suitable for either stern, bow, or side-loading vessels; ferry slips are suitable for bow or sterm loaders; marginal wharves are suitable for side-loading vessels, but can be used for stern and bow-loaders if off-shore structures or pontoons are added to support a transfer bridge; an off-shore berth, consisting of breasting and mooring dolphins, may be

constructed in deep water where dredging would not be needed, however, this type of facility usually requires the construction of a long over-water approach. The berth should have sufficient length, width and depth under all conditions of tide and current for the safe docking and undocking of the largest ships which are expected to use the terminal. The following minimum single ship berth dimensions are desirable to provide adequate space for the berthing and mooring of vessels:

Length - Length of vessel plus beam (minimum).
Width - Beam of vessel plus allowance of 100 ft. as necessary for tug maneuvering.
Depth - Loaded draft of vessel plus allowance of approximately 2 ft. for lower low tides and uneven vessel trim, plus 2 ft. bottom clearance.

The transfer bridge used to connect the ship and the shore at a roll-on roll-off terminal may be single or multiple span with intermediate supports hinged at inshore end and moveable vertically at outshore end to meet the loading deck of the vessel. Outshore end and intermediate supports, if any, can be held and elevation adjusted by any of several methods, including cables attached to overhead winches, hydraulic pistons or pontoons with adjustable ballast tanks. Length of bridge is determined by the tide range of the port, the elevations at light and loaded draft of the loading vessel to be served and the allowable grade on the bridge. A transfer bridge should be long enough to serve an empty vessel at the average yearly highest tide and a loaded vessel at the average yearly lowest tide. Many of the existing and proposed roll-on roll-off vessels are loaded on a single deck, with transfer of units to other decks within the vessel by internal ramps or elevators. For these ships a single-level transfer bridge can be used, although at ports having tidal ranges in excess of about 15 ft., a multi-level transfer bridge might be considerably shorter and less costly. The upper level of a multi-level bridge could be used for the higher tide elevations and the lower level for the lower tide range. For vessels designed for direct loading to several decks, a multi-level bridge is necessary.

The transfer bridge at a trailership terminal is not fixed to the vessel and a sliding plate is used to span the gap between the bridge deck and the vessel deck. The bridge may be of girder or truss construction having a solid or open grid-type deck. Bridge grade should be limited to 10% to minimize the difficulty of backing trailers over the bridge. While the width of the bridge depends upon the size of the vessel's loading ports, a minimum of 20 feet is desirable for a two-way operation and 12 feet for a one-way operation. The minimum vertical clearance should be 14 feet to conform with the design standards of the American Association of State Highway Officials (AASHO). A minimum of 60 feet beyond the inshore end of the bridge is required for maneuvering the recommended size tractor-trailer, with additional clearance for passing traffic. In general, the bridge should be designed for the standard AASHO H20-S16 loading, or for greater loads at ports where increased axle loadings are permitted.

The transfer bridge at a trainship terminal should be designed to support a continuous line of maximum weight (75 ton) cars on each track. Since the vessel must be fixed to the transfer bridge to keep the tracks in alignment, consideration must be given to the loads which will be placed on the bridge by vessel movement under the action of tide, wind and current. Idler cars should

be used between the locomotive and the cars being pushed onto the vessel so that the locomotive does not travel on the bridge. With an allowance for vessel trim, the transfer bridge grade should be limited to 6 percent. The track spacing and number of tracks that should be provided on a transfer bridge depends, of course, on the size of the vessel's loading port and the track arrangement on the loading deck. The vertical clearance should be at least 16 feet to conform with the minimum clearance requirements of the American Railway Engineering Association (AREA).

The area required for marshalling is determined by the size of the vessel, the frequency of sailings and the shape and arrangement of the available area. When trailers or rail cars for only one ship will be in the marshalling area at any one time, and ships can be expected to take on and discharge a full load, space should be provided, at a minimum, for the storage of land carrier units equal to twice the ship's capacity. This permits the discharging of a full shipload of units before beginning the loading operation. When trailers or rail cars for more than one ship will be in the marshalling area at one time, studies of the arrival and departure pattern of the trailers or rail cars are required to determine the space needed.

The marshalling area for a trailership terminal should be surfaced with a high-type pavement, such as concrete or asphalt, with adequate structural strength. Unit parking depth should be limited to two trailer lengths, parked at either 90^{o} or 45^{o}, to obtain accessibility. About 1000 square feet per trailer should be used for planning purposes, including provisions for unusable space around the perimeter of the area and at entrances and exits.

The marshalling area for a trainship terminal is a yard having rail connections to the transfer bridge and to the receiving facility. These connections should be made by a ladder track at each end of the yard. A passing track should be provided within the yard so that switching locomotives may operate at either end. For planning purposes, and based upon a 44-foot car and 13.5-foot track spacing, about 900 square feet per car should be allowed for the marshalling yard, including provisions for a passing track and ladder tracks.

The classification of trailers and rail cars for weight, also size if required, is performed during the transfer of these units from the receiving facility to the marshalling facility. I have previously indicated the characteristics of receiving facilities for both trailership and trainship terminals.

Lift-Off Lift-On Terminals

At lift-on lift-off terminals units are transferred between ship and shore by crane. The units may be trailers, rail cars or containers. At terminals where the tidal variation is great and a transfer bridge would be abnormally long, lift-on lift-off operations are more suitable than roll-on roll-off operations. Ship berthing facilities for lift-on lift-off operations may be slip-type wharves, finger piers or marginal wharves. The cranes may be ship-mounted or shore-based and may be of either the revolving or horizontal boom type. The use of a ship-mounted crane permits a vessel to serve any terminal having a wharf apron suitable for truck or rail operations. The marshalling area, receiving area and subsidiary areas for lift-on lift-off terminals are generally the same as for a roll-on roll-off terminal. The wharf apron for a lift-on lift-off trailer terminal should be at least 40 feet

wide to provide for two lanes of highway traffic and to allow space for the crane wheels and for ship mooring facilities. The crane should be capable of handling loads of at least 25 tons to conform to AASHO standards or heavier loads where greater axle loads are permitted. At trainship terminals, where cars are pulled onto a track-equipped cradle, which cradles are thence lifted by crane via a "well" opening in the ship to an appropriate deck, the wharf apron should be at least 50 feet wide to provide for the cradles and adequate clearances. The crane should be capable of handling loads of at least 75 tons plus the weight of the cradle.

Container Terminals

At a lift-on lift-off container terminal, outbound containers are brought to a receiving facility on highway trailers or railroad cars. They may be removed from these units and placed in a marshalling area by mechanical handling equipment such as fork-lift trucks or straddle carriers. For ship loading, the containers may be brought from the marshalling area to the wharf apron by similar cargo handling equipment. At terminals now in operation where demountable trailer body containers are handled, the container remains on the highway trailer chassis in the marshalling area and is moved to the wharf apron by a highway tractor.

The containers are usually equipped with lifting rings to which the crane hooks are attached. The crane lifts the containers and places them on board the ship. Cargo handling equipment may also be placed on board the ship to move containers to other decks via internal ramps. Inbound containers are unloaded by the crane and placed on the wharf apron for transfer to the marshalling area by cargo handling equipment. The width of the wharf apron is determined by the size of the containers being handled and the type of equipment used, and should be adequate for two lanes of traffic.

Conveyor Terminals

Conveyor operations are an adaptation of the roll-on roll-off principle to containers. Conveyor-type equipment is used for handling containers that are equipped with small flanged wheels. The conveyor may be similar to a car puller, consisting of a chain or cable located between rails on which the containers' wheels roll and equipped with fittings which connect with the containers. The design of such a loading and unloading device must be coordinated closely with the method by which the containers are stowed in the ship. Conveyors can be used for stern, bow, side, or hatch loading, and any of the five ship berthing types can be used with appropriate conveyor systems.

A marshalling facility and a receiving and delivery facility are also needed at a conveyor terminal. The area requirements for these facilities are influenced by the sizes of the containers and types of handling equipment used.

Finally, roll-on roll-off and lift-on lift-off operations for either trailers or rail cars could be combined at a single terminal with joint use of the marshalling and receiving and delivery facilities. At such a terminal, both a transfer bridge and a crane would be required. Also, for the most efficient operation of a roll-on roll-off, lift-on lift-off, or conveyor type service, terminal facilities should be specially designed and built. However, it may be desirable to modify an existing general cargo terminal for use by specialized

vessels and conventional general cargo ships. A major advantage of these special—purpose ships is the saving of in-port vessel time made possible by the use of specialized facilities. To be effective, frequent service of this type must be offered by the shipping line. There would be no advantage in providing a terminal with the facilities needed for one or more of the specialized operations if the berths at the terminal are unavailable for 6 to 10 days because of conventional general cargo operations. If, however, the occupancy of the berth by general cargo vessels ranges from 1 to 3 days and ship schedules can be arranged to eliminate conflicts, it might be feasible to combine general cargo operations with one or more of the specialized operations.

In closing, may I advise that the study now being made by the Maritime Administration of shoreside facilities for trailership, trainship and containership services will be available for public distribution in the next few weeks. It will contain schemes both narratively and graphically presented, as well as layout plans, technical analyses, and design criteria covering the various terminal facilities necessary for these specialized vessel operations. Those of you who may be interested in this new development are urged to obtain a copy for your information and guidance.

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EVOLUTION OF THE MISSISSIPPI VALLEY FLOOD CONTROL PLAN

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(Proc. Paper 1251

ABSTRACT

This paper traces development of lower Mississippi Valley flood control from its beginning with local levee construction at New Orleans to the comprehensive plan now being prosecuted by the Corps of Engineers, U. S. Army. Flood control methods, proposed for controlling the Mississippi, are analyzed in the light of present knowledge.

INTRODUCTION

The control of floodwaters is a complex problem, not always susceptible to a precise engineering solution and yet requiring a high order of professional skill and ingenuity. It is a problem that has been or is being faced in almost every river valley in the United States. In each location the problem is different, because of the volume of flow involved, the degree of settlement of the flood plain, the topography of the basin, or for any one or a combination of many factors, each of which may exert a compelling influence upon the method of attack used in solving the problem.

In no river valley of this country is the problem more complex than in the lower Mississippi River Valley.

The Lower Mississippi River

The lower Mississippi River drains an area of 1,245,000 square miles, including all or parts of 31 States and two Canadian Provinces. See Figure 1.

The average annual mean discharge at Vicksburg, Mississippi, is 595,000 cubic feet per second. The maximum discharge ever measured in the Mississippi River proper was 2,159,000 cfs at Arkansas City in 1937.

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However, had the 1927 floodwaters been confined by levees at this point, it is estimated that the discharge would have been 2,472,000 cfs.

Mississippi River Flows

Flow in the lower Mississippi River is derived in large part from its tributaries. Figure 2 shows the contribution of each major subdivision to the entire drainage basin in terms of percentage of drainage area, percentage of total rainfall, and percentage of runoff.

Of interest is the fact that the Ohio River Basin, while only 16 percent of the total drainage area and receiving only one-quarter of the rainfall, contributes 40 percent of the average annual runoff above the latitude of Red River Landing. The Missouri River Basin is 43 percent of the drainage area and receives 30 percent of the basin's annual rainfall, but it contributes only 12 percent of the runoff above the latitude of Red River Landing. Also of interest is the area designated No. 7, roughly the lower Mississippi River Valley. This area is 7 percent of the total and receives 12 percent of the basin's rainfall. Its contribution to runoff, however, is large, as it makes up 17 percent of the whole. It is necessary to have an understanding of this before a thorough analysis may be made of the flood problem in the lower Mississippi River Valley.

The unusually large drainage area, with its wide variations in climatic, topographic, and cultural conditions, creates a most difficult problem in arriving at a satisfactory solution to the control of floodwaters in the alluvial valley of the Mississippi River.

The Alluvial Valley of the Lower Mississippi River

The alluvial valley of the lower Mississippi River is that broad, gently sloping lowland which begins at Cape Girardeau, Missouri, and extends to the Gulf of Mexico, more than 600 miles in length. This lowland is bordered by abrupt escarpments, the width varying from 30 to 125 miles. Tributary flow joins the main river at various locations, and the lowland extends up these tributaries many miles in addition to forming broad flat basins in the vicinity of the junction. These tributary basins and the lowlands along the main river have in recent geologic times been subject to flooding as the river stages exceeded bankfull conditions.

The area, historically subject to overflow from Mississippi River and tributary floods, is that usually referred to as the "alluvial valley" and is the area to which authorizing legislation refers in defining the scope of the "Mississippi River and Tributaries Project."

The valley is truly alluvial from a geological point of view, and is considered to have been formed during the final cycle of glaciation sequences. During the last glacial stage, when sea level was much lower than at present, the lower Mississippi River Valley became deeply eroded and entrenched between the valley's walls. As the glacial ice mass melted and sea level was raised the valley filled, the gravels and coarse sands being found generally in the lowest horizons. The present alluvial plain consists primarily of sands and silt, grading into very fine sands and silt progressively from north to south. As is typical of rivers carrying a large amount of alluvium, the lower Mississippi developed a highly sinuous course, creating meander loops and

bends and shifting its channel from time to time so that the alluvial plain has been worked and reworked by the river many times. We find, therefore, a complex soil structure in the valley; the materials are easily eroded by the river in its present pattern, and the improvement and control of the river for navigation or flood control is greatly influenced by the nature of the alluvial plain.

As is characteristic of alluvial streams, in time of flood the river deposits sediment in the overbank areas, dropping the greater part of its load adjacent to the stream. For this reason, the banks are generally 10 to 15 feet higher than the lowlands within the tributary basins, causing drainage to be directed away from the Mississippi River and entering the river through tributary mouths. The drainage pattern is further complicated by the similar behavior of tributary streams, which are also alluvial. This basic complication must be kept constantly in mind when developing a flood control plan for the basin.

Development of Plans of Control

The present flood control plan came into being in 1928, following the flood of 1927. However, it has been subjected to much study and subsequent modification, and it is presently being restudied.

The development of a plan for controlling floods must come from a careful and logical analysis, using known engineering methods and techniques. It is necessary to proceed logically, determining first what forces are being dealt with. Following this, it is possible to determine the magnitude and combination of engineering works required. Thus, it is not possible to start with the hypothesis that levees only, or reservoirs only, or floodways only, or even combinations of these elements, will control floods. One must first determine where and in what quantity the floodwaters will occur. It is then possible to examine the elements of flood control to determine which ones and in what combinations they can be used to control the determined flows.

Determination of Design Flood Flows

To produce a major flood on the lower Mississippi River, a sequence of major storms is required, usually extending over a period of one to two months or longer. Because of the large drainage area involved and the relatively short period of record, it is reasonable to assume that all critical combinations of storms and runoff events have not been observed. This was the basic approach to a study of design flood flows recently made by engineers of the Mississippi River Commission in a reexamination of the flood control plan.

About 35 storm combinations, selected by the U. S. Weather Bureau, were studied, including storms of record occurring since the last review of the project. These combinations had extreme variations in rainfall and runoff characteristics and contributions from major tributaries. Of these, four were selected for detailed study to be used as bases for determining magnitudes of the Lower Mississippi River Project Design Flood. These four combinations represent the maximum contributions that would be expected from the major tributaries. Storm combinations and the critical arrangements of the four hypothetical floods are as follows:

Hypo-Flood 52A (late spring season) consists of the combination of one transposed storm and two storms as actually occurred.

Hypo-Flood 56 (early spring season) consists of the combination of two storms as actually occurred.

Hypo-Flood 58A (winter season) consists of the combination of one storm as actually occurred, one storm as actually occurred increased ten percent, and one transposed storm.

Hypo-Flood 63 (early spring season) consists of two storms as actually occurred and one transposed storm.

The four hypo-flood hydrographs were developed from combinations of events that are considered plausible from a meteorological viewpoint and have reasonable probability of occurrence, judging from past flood and storm sequences. It is recognized that there is a probability of the occurrence of unusual combinations of meteorological and hydrological events that could produce a flood of a much larger magnitude than any one of the four selected hypo-floods. However, the occurrence of such a sequence would be a rare probability. The four selected hypo-floods are believed to represent the maximum flows that could reasonably be expected to occur. This is the basic concept of planning for the flood control features of the project.

In general, Hypo-Flood 52A assumes the maximum component that could be expected from the upper Mississippi and Missouri River Basins, Hypo-Flood 58A represents the maximum expected from the Ohio River Basin, and Hypo-Flood 63 represents the maximum from the Arkansas-White River Basin. Hypo-Flood 58A has, in general, the same characteristics of the more frequent floods of record in which the maximum contribution has been from the Ohio River Basin, with moderate contributions from the upper Mississippi and Missouri River Basins and relatively large contributions from the other major tributaries.

Following the determination of the four hypo-floods, each was routed down the Mississippi River by a detailed method of flood routing to determine the resulting daily flows at St. Louis, Mo.; Cairo, Ill.; Memphis, Tenn.; Helena, Ark.; Vicksburg and Natchez, Miss.; and latitude of Red River Landing, La.

Modified hydrographs were then obtained for the hypo-floods for two sets of conditions: with "Group E" reservoirs operating (reservoirs built and reservoirs under construction); and with "Group EN" reservoirs operating (Group E reservoirs plus reservoirs scheduled for construction in the near future).

Thus, hydrographs were determined for key stations within the alluvial valley for floods without reservoirs operating and for floods with two different combinations of reservoirs operating. With these basic data established, it is possible to make a sound reappraisal of the flood control plan.

Such was the method used recently in reviewing the design flood flows for this project. Similar methods have been used in the past. Development of the 1928 flood control plan and the 1941 revision was, in fact, based upon such a study. Following this, it remained, then, to determine the flowlines for such a design flood and to select the engineering methods available for containing such a flood. In addition to the analytical procedures for flow line determination, hydraulic model techniques were used to verify the results.

Original Study of Flood Control Plans

Before the 1928 flood control plan was authorized by the Congress, hearings were held by the Flood Control Committee of the House of Representatives, during which voluminous testimony was presented advocating various plans or combinations of plans of improvement. At this same time, earlier proposals were reviewed in an effort to arrive at the best possible plan of improvement.

For many years the efforts to control floods on the Mississippi River have been the subject of thorough study and analysis. The first study of great consequence resulted in the monumental report of Humphreys and Abbott(1) in 1861. Since that time, many other engineers have analyzed flood control techniques. In a paper presented before the American Society of Civil Engineers in 1917,(2) Colonel C. McD. Townsend grouped flood control methods into six major categories: (1) Methods of reducing extreme variations in runoff by increasing the soil absorption of rainwater, including reforestation, soil drainage, variations in methods of plowing, the construction of small dikes in agricultural areas to retain on the soil a large portion of the rainfall until there is time to absorb it; (2) reservoirs and detention basins which will retain the crests of the runoff and discharge them at lower stages; (3) barriers thrown across mountain streams to prevent erosion and the transportation of debris; (4) channel improvement, cutoffs, and the construction of auxiliary channels; (5) levees to confine the floodwaters to the channel; and (6) improved or new outlets for the purpose of facilitating the discharge of the streams.

Each of the measures listed has had staunch advocates as well as vociferous opponents. Time and space do not permit an exhaustive analysis of each of the measures, but a brief discussion is essential to an understanding of the final plan of improvement adopted.

The effects of forest growth in assisting to hold back runoff are well known. As a primary means of flood prevention, however, there is no evidence to support the advocacy of reforestation to the exclusion of any other measure. It should be remembered that when Hernando DeSoto first viewed the Mississippi River, in 1541, the river was in extreme flood. At that time, heavy forest growth existed throughout the entire Mississippi River drainage basin.

Soil drainage, contour plowing, and the construction of small dikes on agricultural lands do assist in controlling floods to the extent that they hold back floodwaters in the immediate area in which they are located. Advocates of these measures have been most vocal in their insistence that the measures they supported should be adopted as the primary means for controlling floods. Yet, it was apparent in 1927 that the problem of Mississippi River flood control was far too great to be solved by any one measure and that upstream preventive measures, while desirable control measures, would exert but a minor influence on the over-all solution to the problem.

Even in Colonel Townsend's day, he saw the limited contribution to flood control through upstream improvements. In discussing reservoirs and detention basins, he(3) concluded: "At the headwaters of streams storage reservoirs and detention basins can be successfully employed to reduce flood heights. Which method is preferable is dependent upon local conditions. Their efficiency, however, rapidly diminishes as the distance from them increases."

The Nelson Committee's Report,(4) a report of the Senate Committee on Commerce, concluded in 1898, after a study of various flood control measures and the value of reservoirs for full control of floods, that " . . . the scheme (of reservoirs) is regarded by nearly all engineers and other experts as wholly impracticable; in short, your committee can discover no sure or adequate relief in reservoirs."

This theory had been supported earlier by Humphreys and Abbott,(5) who found that "The impracticability of the scheme requires no further demonstration. * * * The idea that the Mississippi delta may be economically secured against inundation by such dams has been conclusively proved by the operations of this Survey to be in the highest degree chimerical."

The subject of cutoffs caused equally as much discussion, with probably as much difference of opinion. Humphreys and Abbott,(6) for example, concluded that "The country above the cut is therefore relieved from the floods only at the expense of the country below. Moreover, if a series of cutoffs were to be made extending to the mouth of the river, the principles educed show that the height of the floods would be regularly decreased from a point near midway of the series to the upper end, and regularly increased from the same point to the lower end. The system, therefore, is entirely inapplicable to the Mississippi River, in whole or in part."

Colonel Townsend, in an address at Memphis, Tennessee, on September 26, 1912, while summarizing the various methods of flood control that had been proposed, concluded that cutoffs afford relief at one locality but at the expense of another. This was in general agreement with the earlier findings of Humphreys and Abbott.

It must be noted that subsequent developments have proven even these distinguished engineers to have been incorrect in their conclusions regarding the downstream effect of cutoffs. Their views concerning the limited usefulness of upstream reservoirs to provide flood control in the lower valley have, in general, been proven to be sound, but we now find that there is a substantial benefit and reduction of flood crests resulting from reservoirs built and planned on tributaries primarily for benefit of the tributary area.

As to outlets for the floodwaters, Humphreys and Abbott(7) discussed this proposal at great length. They recognized the desirability of having a secondary outlet for floodwaters, but found that "Unfortunately, the relief of the river itself is only half of the difficulty. The water taken from it still remains to be disposed of. A channel to conduct the water must then be prepared. Here lies the great practical difficulty which renders the system of comparatively little avail for protecting Louisiana against overflow."

At the time their report was rendered (1861), Humphreys and Abbott were undoubtedly on firm ground in coming to such a conclusion. The Atchafalaya River, now the Mississippi's largest and most useful outlet, had not then developed to the extent that it could seriously be considered as an outlet for floodwaters of large magnitude.

Mr. Charles Ellet(8) also gave much serious discussion to the many plans for controlling Mississippi River flood. In summarizing his findings, he recommended a comprehensive plan which included, in addition to levees and reservoirs and the prevention of cutoffs, a recommendation for the " . . . formation of an outlet of the greatest attainable capacity, from the Mississippi to the head of Lake Borgne at New Orleans, with a view, if possible, to convert it ultimately into the main channel of the river."

Outlets were also discussed by Colonel Townsend,(9) who stated a belief

that the effect of outlets in reducing flood heights is not as great as popularly supposed. He based his questioning of the practicability of outlets upon the fact that the outlet had to be controlled to be effective and upon the difficulty in regulating the velocity with which the water will flow through the outlet at varying heights of the main stream. "If it is so constructed that it will discharge at a greater velocity than the river itself," he concluded, "there is danger of its enlargement to such an extent as to divert the greater part of the flow down it and transfer the main stream itself into an outlet; and, if, on the other hand, it discharges at a lower velocity, it will tend to fill with sediment." This was actually the basic concern which faced the Mississippi River Commission by the indicated diversionary forces so active in recent years at Old River which led to the current construction of control structures and the closure of this natural distributary into the Atchafalaya Basin.

Thus, the discussion of the plans by which Mississippi River floods might be controlled continued. Meanwhile, there was a large group of engineers and laymen who supported strongly, and often to the exclusion of all other means, a great levee system designed to confine the floodwaters and prevent their overflowing the valley lands. It was generally agreed that levees would have to play a predominant role regardless of other features that would be worthy of incorporation in the over-all plan.

Colonel Townsend(10) said of levees that "On the lower alluvial reaches of long rivers, such as the Mississippi and Colorado, they afford the only sure means of flood control."

It is evident that, in the seventy-five years preceding the adoption of the present plan, the problem of controlling Mississippi River floods received most careful study and much thought. Not only engineers, or local interests to whom the problem was personal, took an interest in this. There were many others who advanced solutions; some of these had doubtful application.

Colonel Charles L. Potter, President of the Mississippi River Commission, stated in 1925 that "There is more wasted ingenuity, unsupported by technical knowledge, applied to the control of floods in the Mississippi River than to any other problem in the United States. The public thinks it an unsolved problem, and expects that Yankee ingenuity, rather than deep study along the lines of hydraulic engineering, is going to hit upon a solution."

Writing in the Engineering News-Record of April 2, 1925, Colonel Potter listed briefly some of the theories to which his attention had been directed.

"One individual offered to carry a depth of 100 feet from the Gulf to St. Louis by directing the strong surface current to the bottom and flushing out all sediment. How the inversion of the river was to be accomplished has never been developed."

"Another suggests that the bottom current, which contains the greater part of the sediment, be conducted away in pipes."

"Just now a gentleman in Michigan has a scheme whereby floods may be reduced 'any amount desired.' When approached as to the nature of his plan, he declines to divulge it."

So, the engineers who grappled with the complex and most perplexing flood control problem were not withou suggestions as to how the job might be accomplished. Now were they without recommendations regarding the practicability of the various flood control features.

It was clearly evident, after the flood of 1927, that strong national action had to be taken. After exhaustive study, the plan of the Chief of Engineers (known as the Jadwin Plan, so named from its author, Major General Edgar

Jadwin, then Chief of Engineers) was adopted by Congress in the Flood Control Act of May 15, 1928.

Development of the 1928 Plan

The Jadwin Plan was truly a comprehensive one. It provided for control of the maximum flood which meteorologists believed possible of occurrence, and it provided for future expansion to meet changing conditions. It included a spillway above New Orleans, diversion floodways in the Atchafalaya and Tensas Basins, a riverbank floodway from Cairo, Illinois, to New Madrid, Missouri, together with strengthening and a moderate raising of existing levees. It was designed to prevent any material increase in flood stages. Channel stabilization and navigation improvements were also included.

The basic act of 1928 was essentially a plan for the control of main stem flood flows of the Mississippi River in the alluvial valley. It was pointed out in the Jadwin Plan that recommendations for the tributary basins could not be prepared in the limited time available for the formulation of the main stem plan and plans for improvements in the tributary basins would be submitted separately. Subsequent legislation has incorporated tributary features into the project, thereby extending the benefits from flood control and making the development of the valley truly comprehensive.

Thus, the basis for the present flood control plan came into being in 1928. It was promptly subjected to a searching analysis. A very exhaustive report was made in 1931, as a result of a resolution adopted by the Flood Control Committee of the House of Representatives. This report examined at great length a number of suggested plans of improvement, each of which was found to be more expensive or less effective than the plan adopted by the Act of May 15, 1928. The Board of Engineers for Rivers and Harbors and the Chief of Engineers, reporting upon the various plans considered during the study, recommended that no modification of the adopted project be made. Again, it was concluded after thorough study that the complete control of floods in the lower Mississippi by reservoirs alone is impracticable.

Among the recommendations of the Board of Engineers for Rivers and Harbors was one for experimental work to ascertain the feasibility of increasing the capacity of the main river channel by a well coordinated plan including channel rectification and stabilization, dredging and bank protection, beginning at the mouth of Red River and working gradually upstream. This laid the foundation for the program of cutoffs undertaken in 1932.

Progress on construction of the features called for in the 1928 Act was rapid. Main river levees were closed and strengthened in time to withstand the 1929 high water, and other works were undertaken and carried forward at a rapid rate. Thus, it soon became apparent that the plan of control was a sound one. But technical sufficiency is not the only evaluation of a great project where the holdings and livelihood of many people are involved. The justification for subjecting certain floodway areas to the degree of servitudes resulting from the basic law was questioned as the economy of the lower valley began to recover. In response to a Congressional resolution directing review of the plan, the Chief of Engineers submitted a report(11) to Congress on February 12, 1935. A bill incorporating the recommendations of this report was enacted into law and approved by the President on June 15, 1936. This Act amended the 1928 Act to provide for modifying the project insofar as

floodways were concerned, for improving the Atchafalaya River and its outlets, for partial protection in the White River backwater area, and for headwater flood control in the Yazoo and St. Francis tributary basins. It also included provisions for elevated crossings of highways and railroads over floodways, for drainage works made necessary by levee construction, and for levee access roads.

Meanwhile, active construction work continued on the various features of the plan. Levees were raised and strengthened. Work progressed on the floodways and outlets in the Atchafalaya Basin. And a program of cutoffs was initiated and carried to completion; sixteen cutoffs were made under the direction of the Mississippi River Commission, reducing the river distance between Memphis and Baton Rouge by some 170 miles. All of these works exercised a marked effect upon the effectiveness of the flood control plan.

During the 1937 flood, the completed works were put to another severe test and proved the soundness of the plan. The success of the cutoffs was effectively demonstrated, even with flows above Arkansas City greater than in 1927. Flood stages were lowered by 8 to 10 feet at Greenville and 12 to 14 feet at Arkansas City, without adverse effects on stages downstream from the cutoffs.

In 1939 and in 1940, resolutions were adopted by Committees of the House of Representatives and of the Senate requesting the Chief of Engineers to again review the entire project and to submit recommendations for any desirable modifications.

In response to these resolutions, the Chief of Engineers submitted a report[12] on April 2, 1941, setting forth several engineering plans both with and without floodways in the middle section, but did not recommend any specific plan. After extended hearings, the Congress enacted the Flood Control Act of August 18, 1941, which called for the abandonment of floodways in the middle section, primarily because of the demonstrated effectiveness of the cutoffs; provided for raising main river levees to confine the project flood; added improvement of the Yazoo and Red River backwater areas; and made other minor modifications to the plan.

Since 1941, numerous other additions and modifications have been made to the 1928 plan. Among these are extensions of works in the tributary areas, the provision of drainage works, and the addition of local protection projects. In 1954, the project was further amended to provide for the control of flow at Old River and the Atchafalaya River to prevent diversion of the Mississippi River through the Atchafalaya Basin.[13]

Description of Present Plan

It is apparent that the present plan for flood control and navigation improvements in the lower alluvial valley of the Mississippi River has gradually evolved, through amendment and modification, from the basic plan approved by the authorizing legislation in 1928. It is still the same basic plan, but it has been modified to meet changing economic, physical, and engineering conditions. There is no doubt that it will be modified still further as conditions change in the future. This is one of the outstanding characteristics of the plan, and certainly one of its most admirable features.

This plan is designed to control a flood even greater than that of 1927. The design, or "project," flood is the greatest flood which meteorologists

believe can reasonably be expected to occur. It assumes a flow at Cairo, Illinois, of 2,450,000 cfs. (Without reduction by the headwater reservoirs on the upper Mississippi, the Missouri, and the Ohio Rivers and their tributaries, the flood flow at Cairo would be 2,600,000 cfs.) Below the mouth of the Arkansas River, the project flood is from 17 to 27 percent greater than the flood of 1927, amounting to 3,000,000 cfs at the latitude of Old River, just below Natchez, Mississippi.

The flood control plan is a blending in carefully selected quantities of a number of proven features, each of which performs a contributing function and all of which together, when completed, will protect the alluvial valley from the project flood.

Major features of this truly comprehensive plan are reservoirs on the tributary streams, both within and without the alluvial valley, to hold back flood flow insofar as practicable; levees, on the tributaries and on the Mississippi River, serving to confine the flow to a carefully designed leveed channel and backwater areas; cutoffs, on the Mississippi River, to speed flow down the river and lower flood stages; revetment, which is placed to protect flood control structures and to assist in stabilizing the channel; and overbank floodways, which divert flow from the river. The plan also makes use of dikes, pumping plants, floodgates and floodwalls, and other features. Each of these makes an important contribution to the proper functioning of the comprehensive plan of improvement.

It is readily apparent that each of the features which have been combined into the comprehensive plan has been subjected in the past to both ardent praise and violent criticism, advocated to the exclusion of all others, or condemned as being unworthy of consideration. This comprehensive plan bears out the statement made by Mr. Kenneth C. Grant, who, in writing on the subject of flood prevention,(14) in 1917, said that "In many cases no one method may provide an adequate solution to the flood problem, but the best plan will consist of a combination of two or more methods. It may be found unfeasible or uneconomical to reduce flood flow to existing channel capacities by reservoir or detention-basin control. It may be found equally unfeasible or uneconomical to provide an adequate channel to accommodate maximum floods. Protection by one method is best carried out to the point where the remaining necessary protection is cheaper by some other method."

Protection against overflow by the main Mississippi River is accomplished by the construction of levees and appurtenant flood outlet structures and floodgates. See Figure 3. The levees extend from high ground at the head of the several basins, into which the valley naturally divides, to the major tributaries which normally form the lower boundaries of the side basins. There are 1,599 miles of main line levees and 1,955 miles of smaller embankments on the tributaries and floodways and backwater areas.

The levee line on the west bank begins just south of Cape Girardeau, Missouri, and, except where the waters of the St. Francis, the Arkansas-White, and the Red join the Mississippi, extends unbroken to the Gulf of Mexico. The east bank of the river is protected by levees alternating with high bluffs.

From Commerce, Missouri, south the west-bank levee line is continuous to the mouth of the St. Francis River just above Helena, Arkansas. Another levee line begins at Helena and protects the front of the White River Basin. The longest continuous levee line in the project, and probably in the world, begins at high ground near Pine Bluff, Arkansas, and follows the south bank

of the Arkansas River to its mouth, then down the Mississippi to Old River, a distance of 380 miles. The levee line begins again at high ground on the south bank of Red River, a few miles above Alexandria, Louisiana, and, except for a gap at the head of the leveed Atchafalaya River and a stretch of about 11 miles where the Mansura Ridge along the lower Red River is above overflow, runs continuously for 358 miles to its terminus at Venice, Louisiana.

On the east bank, a short levee line 22 miles long begins at Hickman, Kentucky, and extends into upper West Tennessee. Beginning just below Memphis at the head of the Yazoo Basin, there is a continuous 270-mile levee line to just above Vicksburg, Mississippi. The east bank is largely hilly from Vicksburg to Baton Rouge, where the levee line begins again and runs continuously for 172 miles past New Orleans to Bohemia, Louisiana.

Near the mouths of the major tributaries, main line levees are built to project grade, fixed generally to correspond to the assumed peak discharges of the project flood. The project also calls for levees extending upstream on lower portions of the basins of the St. Francis, the White, the Arkansas, the Yazoo, and the Red Rivers at reduced grade to give protection against ordinary floods to lands that would otherwise be covered when overbank stages permitted water to back around the ends of the main line levees. Appurtenant drainage works and some pumping plants provide protection to critical local areas.

Between Cairo and New Madrid, Missouri, the east-bank bluffs and the levee as originally built on the west bank left only a narrow channel through which the river could flow at flood stage. This impeded the flood flow and raised stages. To protect the city of Cairo and to reduce the flood heights, a setback levee was constructed about five miles west of the river-front levee through this reach. The strip between this setback levee and the levee adjacent to the river forms what is known as the Birds Point-New Madrid Floodway, designed to be operated only at extremely high stages. Water enters the floodway through "fuse plugs" in the old front levee opposite Cairo and re-enters the main river just above New Madrid. Operated during the flood of 1937, this floodway was of material aid in reducing flood heights at Cairo.

At the latitude of Old River, 207 river miles above New Orleans, the project flood is estimated at 3,000,000 cfs. The project provides for dividing this flow, with one half continuing down the main river channel and the remainder being diverted to the Morganza and West Atchafalaya Floodways and to the Atchafalaya River. Figure 4.

Of the half flowing down the main channel below Morganza Floodway, 250,000 cfs are to be diverted to Lake Pontchartrain and the Gulf of Mexico through the Bonnet Carre Spillway, located about 25 miles above New Orleans. The remaining 1,250,000 cfs will continue down the river to the mouth.

The Bonnet Carre Floodway was operated in 1937, 1945, and again in 1950. It was very effective in reducing flood stages in the lower reaches of the river.

That portion of the flow diverted from the main channel in the vicinity of Old River is carried by the Atchafalaya River, the Morganza Floodway, and the West Atchafalaya Floodway. The Morganza and the West Atchafalaya Floodways follow down on opposite sides of the Atchafalaya River until the end of the levee system along the Atchafalaya River is reached; there they merge into a single, broad floodway whose flow is drawn off at Wax Lake Outlet and through Berwick Bay into the Gulf of Mexico. The Morganza Floodway

(scheduled to be placed in operation first, the water entering the floodway through a control structure just above Morganza, Louisiana) will carry about 600,000 cfs, and the Atchafalaya River will carry about 650,000 cfs. The remainder of the diverted flow (about 250,000 cfs) will be carried by the West Atchafalaya Floodway, entering through a fuse plug levee at the head of this floodway.

A vital part of the present plan is the program for stabilization of the river banks. This is accomplished by means of revetment, dikes, and dredging. The work is not continuous, but is constructed at critical locations where excessive bank caving threatens main line levees, highly developed areas, or induces unfavorable channel alignments. The stabilization works are designed to fix the river in a desirable pattern to prevent destruction of flood control works, to prevent the river from recapturing the mileage taken from it by cutoffs, improve the river for navigation, and provide a more efficient channel for the discharge of floodwaters. The stabilization program is estimated to require a total of about 689 miles of revetment, together with dikes and contraction works.

Headwater flood control on the St. Francis and Yazoo River systems will be provided by reservoirs (Wappapello Reservoir in the St. Francis Basin and Arkabutla, Sardis, Enid, and Grenada Reservoirs in the Yazoo Basin) supplemented by levees and channel improvements.

The improvement of interior streams for flood control and drainage consists of stream clearing, cutoffs, and channel excavation and enlargement. The authorized project has been extended to include work of this character in most of the large tributary basins within the alluvial valley.

It has been demonstrated clearly that flood control and navigation improvements are inseparable on the lower Mississippi River. Many works serve both ends at the same time, and no work is undertaken for one purpose without fully considering what, if any, effect the work will have on the other purpose.

Review of the Plan of Improvement

As has been noted, the plan of improvement has often been subjected to rigorous examination, and one such review is in progress at the present time. The current review covers the work already completed, the results obtained, and the effects upon the flood problems in the lower Mississippi River of improvements and developments in tributary river basins. This review is being made to determine whether the plan as conceived in 1928 and modified throughout the years since then will perform as intended, whether any elements are no longer necessary, whether the weather records and storm occurrences recently experienced warrant any change in the basic assumptions, or whether any additional features must be constructed to meet the needs of the future.

Great changes have taken place in the lower alluvial valley since 1928, resulting in problems which were not anticipated when the plan of improvement was first authorized. There has been a large increase in population and the over-all economy in the valley; there have been very large increases in the demands for water supply; and there have been reductions in wildlife habitat as a result of increasing settlement and development together with concurrent pressures for the development of wildlife habitat. In recent years, large

industries have moved into the lower 200 miles of the river and to a considerable extent above that point, indicating an awareness of the security and opportunities which exist along this great river for a growing America. All these factors have contributed to the need for the present reexamination of the project.

An interim report now being prepared by the Mississippi River Commission will be submitted to the Chief of Engineers about mid-1957. It will cover the main stem features of the project only, including such important works as levees, channel stabilization, minor harbor improvements, and the like. About one year later, the Mississippi River Commission will submit a final report to include the tributary basins of the alluvial valley. This report will also include a study of the benefits of headwater reservoirs, including authorized reservoirs on all major tributaries covering all parts of the Mississippi drainage basin.

CONCLUSION

It is probable that no engineering undertaking has ever been the subject of so much study and examination by so many qualified persons. Literally millions of words have been written on the subject, with an extremely wide diversity of opinion. The present plan of improvement may truthfully be said to have evolved through a gradual process of development, using the best features proposed, after the most thorough study possible. Each of the features has been thoroughly tested and has proved successful. Yet it should not be concluded that the present plan is the final unalterable solution to the problem of controlling floodwaters of the Mississippi River.

The engineering plan has been developed through sound thinking based upon established hydraulic principles. Yet changes in treatment and even in concept are necessary from time to time in order that the project keeps pace with the great economic development occurring in the alluvial valley.

ACKNOWLEDGMENTS

In the preparation of this paper invaluable assistance was received from various members of the staff of the Mississippi River Commission, but I want to mention especially Mr. R. A. Latimer, Technical Consultant, Mr. J. C. Goodrum, Chief of the Technical Liaison Branch, and Mr. E. J. Williams, Jr., Mr. J. E. Sanders, and Mr. E. A. Graves, of the Hydraulics Branch, whose contributions are most appreciated.

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FIGURE 1. THE DRAINAGE BASIN OF THE MISSISSIPPI RIVER

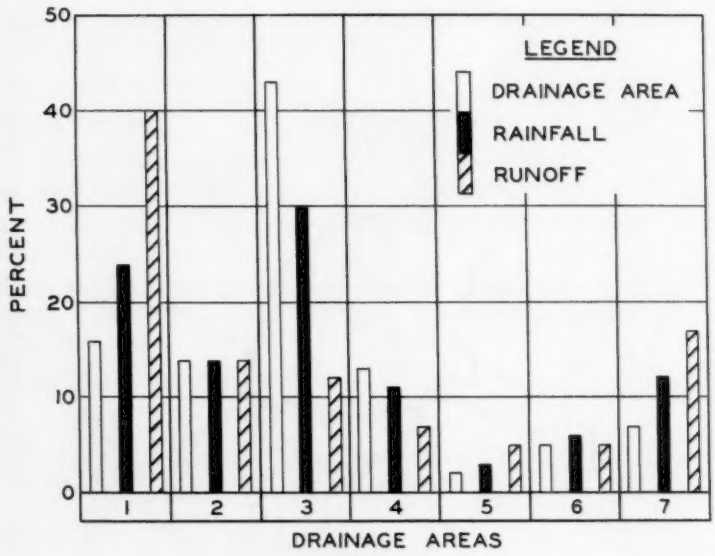

FIGURE 2. A COMPARISON OF DRAINAGE AREA SIZE, AMOUNT OF RAINFALL, AND CONTRIBUTION TO TOTAL RUNOFF.

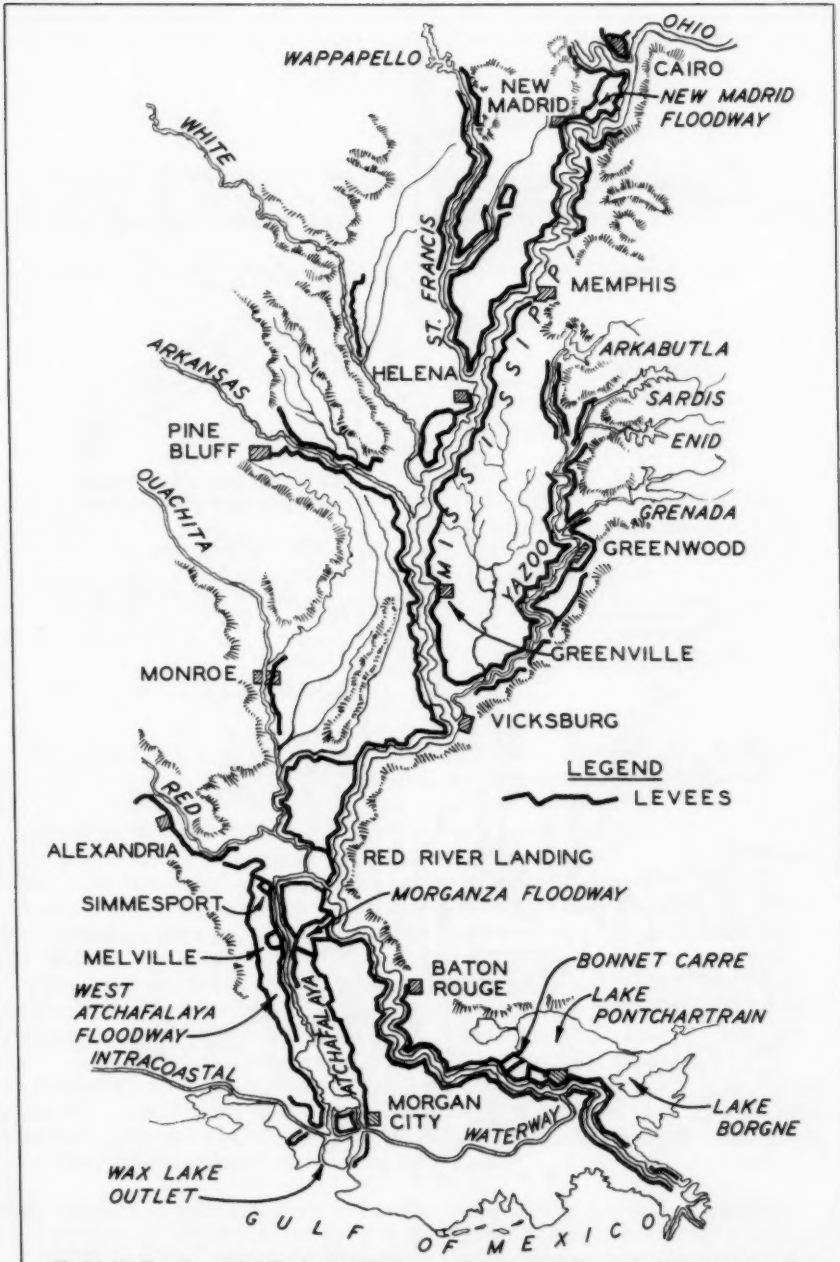

FIGURE 3. THE LOWER MISSISSIPPI RIVER VALLEY FLOOD CONTROL PLAN

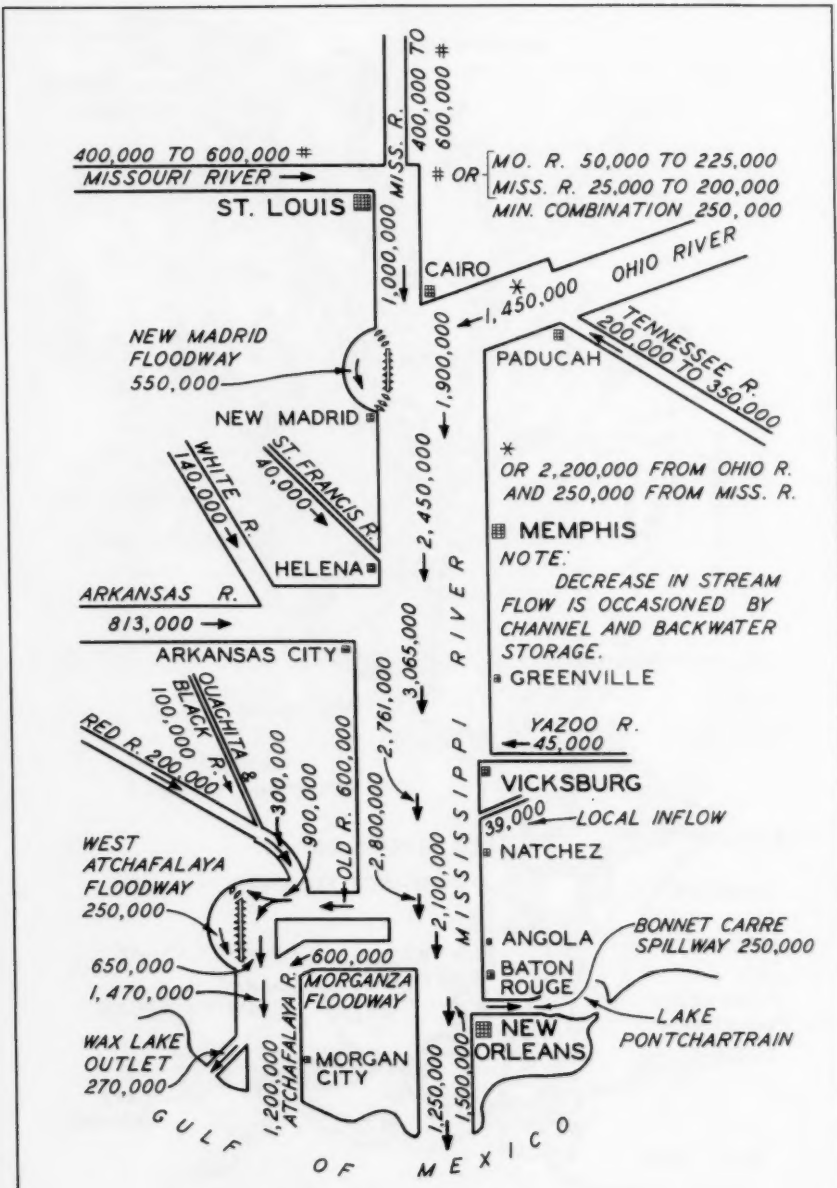

FIGURE 4. DISTRIBUTION OF THE PROJECT FLOOD

Journal of the
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NAVIGABLE WATERWAYS AND INDUSTRIAL DEVELOPMENT[a]

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(Proc. Paper 1252)

ABSTRACT

This paper discusses effects of navigable waterways upon industrial development in the South. The effects of availability of abundant water and protection against flooding are also discussed. Parallel developments of navigable waterways and industrial concentrations are traced for the Mississippi and Warrior-Tombigbee Rivers and the Gulf Intracoastal Waterway.

INTRODUCTION

The deep South is in the throes of a great economic revolution. Its economy, formerly almost totally agricultural, is now moving rapidly toward a balance between agriculture and industry.

The reasons behind this revolution are many, and they are, for the most part, interrelated. Several of the reasons stand out clearly, however. Among these is an abundance of water, which today's industry demands in great quantities for processing and for shipping. The waterways of the deep South meet this requirement with ease. Industries locating in the deep South find well-maintained waterways suitable for the movement of raw materials and finished products in barges or, in some localities, by deep-draft freighters and tankers. These same waterways, supplemented by others which are not maintained for navigation, furnish a vast supply of water usable by industry for manufacturing and processing.

Industrial planners, preparing for the location of plants in the deep South, point to the availability of water as one of their prime reasons. The recently established trend toward waterside location will continue well into the future as industry's dependence upon water increases.

Note: Discussion open until October 1, 1957. Paper 1252 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 83, No. WW2, May, 1957.

a. Presented at ASCE Annual Meeting, Jackson, Miss., February 18-22, 1957.

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History of Navigation Developments

Mr. Roger W. Benedict, writing in the Wall Street Journal(1) recently, called the rush to the rivers a " . . . repetition of early U. S. history. In the 18th and early 19th Centuries, towns and industries huddled near rivers, a major source of transportation, power and even communications."

Practically every great civilization of recorded history has been based upon transportation by water. The civilization of ancient Egypt was essentially that of the Nile River. The Roman Empire depended upon mass transportation on the Mediterranean Sea. And the ability to transport huge quantities of commodities on the waterways contributed immeasurably to the development of the United States.

The present, highly developed system of dependable waterways has come into being through developments extending back for more than one hundred thirty years. In 1820, the first Federal legislation was passed in the interest of navigation, an Act authorizing a survey to be made in connection with navigation on the Mississippi and Ohio Rivers. Four years later, the survey complete, another Act of Congress directed the removal of snags, sawyers, and other obstructions from the channels of those rivers. This marked the entry of the Federal Government into the field of navigation developments on American rivers. Since that time, the Corps of Engineers, U. S. Army, assigned the task of developing America's waterways, has, under Congressional direction, constructed and placed into operation a 28,600-mile-long system of inland waterways, the most extensive in the world, and 300 coastal harbors. A substantial portion of this development has occurred in the deep South.

The lower Mississippi River is one of the most important navigable waterways in the world. From Cairo, Ill., where it joins the 9-foot-deep Ohio River, south to Baton Rouge, La., navigation depths of at least 9 feet and a width of 300 feet are maintained even during the lowest river stages. The authorized depth of navigation channel between Cairo and Baton Rouge has been increased to 12 feet, the 12-foot channel to be obtained by a program of bank stabilization and maintained by dredging. Considerable progress has been made toward realization of the deeper channel.

Between Baton Rouge and New Orleans, the navigation channel maintained is 35 feet deep and 500 feet wide. From New Orleans to the Head of Passes, a 35-foot-deep and 1,000-foot-wide channel is maintained. A channel 35 feet deep exists through Southwest Pass and bar and a channel 30 feet deep exists through South Pass and bar.

The 12-foot-deep Intracoastal Waterway connects with the Mississippi River at New Orleans. It extends westward to the Mexican border, and eastward to Apalachee Bay, Florida. An alternate connection from the Mississippi River at Algiers, La., to the Gulf Intracoastal Waterway is now in use. Another alternate connection from the Mississippi River to the Gulf Intracoastal Waterway is provided by a 9-foot-deep and 100-foot-wide channel from Plaquemine to Morgan City, La. This route is now being enlarged to 12 by 125 feet and is being extended to Port Allen, opposite Baton Rouge, where a new lock is being constructed.

The Black Warrior-Warrior-Tombigbee river system provides a channel 9 feet deep and 200 feet wide from Mobile, Ala., where it joins the Gulf Intracoastal Waterway, to Port Birmingham, a distance of 467 miles.

Along the Gulf Intracoastal Waterway are feeder channels, connecting great port cities to this important waterway. For example, the Houston Ship

Channel extends through Galveston Bay and Buffalo Bayou to Houston. The 50-mile channel is 34 feet deep.

In addition, there are many other important navigable waterways in the deep South, although space does not permit a listing of them.

Evolution in Region's Economy

As the twentieth century began, the South as a whole had only 9 percent of the nation's manufacturing facilities, as compared to the 25 percent it now has. The economy of the region was geared, permanently it seemed, to agriculture and industry closely allied to and dependent upon agriculture. Traveling through the deep South, one saw mile after mile of cotton, corn, sugar cane, and other crops. Industry, when it existed at all, except in the large metropolitan centers, consisted largely of cotton gins, sugar cane mills, and the like. The South knew little of the industrialization, with its far-reaching economic impacts, of other more prosperous sections of the country.

During the past 50 years, this picture has changed almost completely. Agriculture still is the basic element in the economy of the region, but it is sharing, more and more, with industry in a balanced economic structure.

It is not entirely coincidental that the evolution in economic structure has been accompanied by an increase in development and usage of the region's water resources, especially its navigable waterways. Freight carried on the upper and lower Mississippi River systems combined in 1906 (and this includes the tributaries of those streams) was less than 14 million tons; in 1955, more than 94 million tons of important commodities were moved between Minneapolis and the Gulf of Mexico. Commerce in Mobile Harbor grew from less than 3 million in 1906 to approximately 15 million tons in 1955. In 1906, development of the Gulf Intracoastal Waterway was just getting under way, and the Houston Ship Channel was just a dream.

The past fifty years have seen great changes—in the industrial-economic pattern and in the development of waterborne commerce. Today, the banks of the nation's navigable waterways are lined with huge and active industrial plants of all types, sizes, and descriptions. Six and one-half billion dollars worth of new plants were built during 1955 at waterside locations. Huge tows, some even longer than the QUEEN ELIZABETH, hauling as much as 400 railroad cars or 1,000 trucks at one trip, ply the 28,600 miles of navigable waterways. It is obvious that the availability of water—for processing, manufacturing, and shipping—and the growth in industrial development are complementary.

The economy of the deep South has been transformed more radically than any other section of the nation in recent years, and this development also parallels closely the development in water transportation facilities in the area. When announcing a new 60-million-dollar alumina plant at Grammercy, La., recently, Mr. D. K. Rhodes, Vice President of Kaiser Aluminum and Chemical Corporation, said, "The significance of the location of Kaiser Aluminum's new facilities cannot be too strongly emphasized. By locating plants along routes of economic water transportation, a direct flow of raw materials is thus provided." This was stated somewhat differently by Mr. F. L. Linton, Comptroller of Allied Chemical and Dye, which has 45 waterway plants in operation and two more under construction: "In the chemical

industry, particularly in the manufacture of heavy industrial chemicals, the primary advantage of a waterside site is the lower transportation cost afforded by moving both incoming raw material and outgoing finished products in large bulk shipments by barge."

Recent Developments along Navigable Waterways of the South

Gulf Intracoastal Waterway Section

Spectacular advances in industrial development are being made along the Gulf Intracoastal Waterway. A recent issue of Time(2) carried this statement: "At least 500 companies (among them: Reynolds Metals, Alcoa, Monsanto, Dow Chemical) have built plants and warehouses along its banks, while thousands of others use it for cheap transportation. The biggest waterway customer of all is the booming Gulf Coast oil industry, which last year shipped out some 25 million tons of petroleum products, more than half of all the waterway's traffic. Virtually every big company has fields, tank farms, refineries along its banks clear down to Corpus Christi—Texas Co., Standard Oil of N. J., Superior Oil, Magnolia, Kerr-McGee, Pure Oil, Cities Service, Shell Oil, Gulf, Humble Oil. This year (1956) alone 22 companies will either build or expand plants along the waterway. Reynolds Metals is expanding; so is Alcoa, with a new $45 million aluminum plant at Point Comfort, Texas. Estimates are that the surging chemical and petro-chemical industries will shoot up 70% by 1960, and the Gulf Coast will get much of the expansion. Texas alone will add $260 million worth of new plants in the next two years. Firestone Tire & Rubber is building a huge chemical plant at Orange, Texas; Dow Chemical is expanding its Freeport, Texas, plant by $45 million, while Gulf Oil, Foster Wheeler Corp., Lake Charles Chemical Co., Buckeye Cellulose Corp., Coastal Chemical Corp., Union Carbide & Carbon are pouring in millions more for new facilities along the waterway from Florida to Mexico."

Warrior-Tombigbee Basin

New industry is locating and existing industry is expanding along the banks of the Warrior-Tombigbee Waterway at a record rate. In recent years, large expansions have taken place in the chemical, paper, aluminum, foundry, garment, fabric, and other industries.

Much of the chemical growth has been concentrated in the lower reaches of the waterway near Mobile. In 1951, Olin Mathieson Chemical Corporation, first chemical company to enter the area, built a $12,000,000 chlorine and caustic soda plant at McIntosh, located on the waterway approximately 40 miles north of Mobile. With a chlorine supply assured, Geigy Chemical Company, Inc., built a $1,250,000 plant at McIntosh for the production of insecticides. Calabama Chemical Company, now owned by Olin Mathieson, built a $500,000 insecticide plant, and Stauffer Chemical Company opened a $2,000,000 carbon bisulfide plant.

Most recent chemical newcomer to this area is Consolidated Chemical Industries, Inc., now building a sulfuric acid facility at LeMoyne, near McIntosh. Costing $2,000,000, this plant will have a capacity of 500 tons daily.

Earlier this year, Olin Mathieson announced plans for a $7,500,000 expansion. Construction is under way on the new addition.

Caustic soda from Olin Mathieson supplies Courtaulds' $24,000,000 viscose

rayon plant at LeMoyne, about 20 miles north of Mobile. Carbon bisulfide for Courtaulds comes from Stauffer Chemical's plant.

Courtaulds' plant, announced in 1951, has tripled its rayon output. Olin Mathieson expanded to meet increased demands for caustic soda, and Stauffer Chemical enlarged its capacity to keep pace with the growing requirement for carbon bisulfide.

At Demopolis, Ala., the Borden Company's Chemical Division began operating a new polyvinyl acetate unit in August. Capacity of the plant is 10,000,000 pounds annually. Borden's Chemical Division has had a Demopolis operation since 1950, manufacturing formaldehyde and ureaformaldehyde resins.

Production of sulfuric acid and oleum was started in May at a new $800,000 unit of Reichhold Chemicals, Inc., at Tuscaloosa, one of the principal cities along the waterway. In 1954, Reichhold added facilities costing $3,000,000. Central Foundry Company at nearby Holt announced a $2,000,000 expansion last year.

Ketona Chemical Company, owned jointly by Alabama By-Products Corporation and Hercules Powder Company, was formed at Birmingham in 1954 for the production of anhydrous ammonia. This is the first plant outside Europe to base its entire ammonia production on coke-oven gas, which is supplied by Alabama By-Products. Facilities for the production of nitrogen solutions are now under construction.

At Mobile, International Paper Company has begun a major expansion which includes a $20,000,000 newsprint mill with a 100,000-ton capacity annually, an $18,000,000 addition for the manufacture of kraft paper, and $5,000,000 for an office building and other facilities. Late last year, Scott Paper Company announced plans for an expansion costing an estimated $20,000,000 at Mobile. This will ultimately make the Mobile plant the principal source of Scott paper products for distribution in the South.

Another fast-growing Mobile industry which recently doubled its plant investment from $3,000,000 to $6,000,000—National Gypsum Company—is now in full expanded production.

At Naheola, farther upstream, the Marathon Corporation broke ground in July for construction of a $55,000,000 paper mill, pulp mill and converting plant. Gulf States Paper Corporation, which has a major plant at Tuscaloosa, is builting a $25,000,000 paper mill at Demopolis.

Lower Mississippi River

Developments along the lower reaches of the Mississippi River have been as rapid as, and in some ways more spectacular than, those in other river basins of the South.

Between New Orleans and Baton Rouge, a great industrial empire is building up at a spectacular rate. During the past five years, more than 600 million dollars has been invested by industry for new and expanded plants in this area. Major industries are primary metals, chemicals, and petroleum refining. The nation's largest oil refinery, the nation's largest aluminum reduction plant, the nation's third largest sugar refinery, and the nation's second largest alumina plant are located in this area, which also claims the nation's only plant producing building products from bagasse and one of only two plants in the nation producing anti-knock compounds for motor fuel.

Between 1950 and 1953, manufacturing payrolls in this area increased by more than 50 percent.

The most impressive industrial growth for the area has been in the natural gas and petroleum-based chemical industries. During the last five years, eleven new plants or expansions have been constructed. In addition, most of the petroleum refiners along the river have added chemical facilities in conjunction with their refineries.

Another major industry along the Mississippi River is building products. Seven multi-million-dollar new plants and expansions of this type have been completed in the last five years, producing wallboard, insulation board, cement, gypsum products, and floor tile.

Recent announcements of industrial expansions reflect further growth in the area. Among these are a 50-million-dollar plant for Dow Chemical Company at Plaquemine, La., to produce caustic soda, chlorine, and several organic chemicals; an 8-million-dollar plant for the production of ethylene oxide and a 20-million-dollar plant for the production of chlorine and caustic soda, both for Wyandotte Chemicals Corporation, at Geismar, La.; Kaiser Aluminum and Chemical Corporation has announced two plants, one in the industrial canal area of New Orleans for storage facilities for bauxite and a 60-million-dollar facility at Grammercy, La., to produce alumina and caustic chlorine. Crown Zellerbach Corporation has purchased 1900 acres in the southeast corner of West Feliciana Parish in Louisiana, and Olin Revere Metals Corporation has announced plans for the construction of an alumina processing plant to cost $50,800,000 at Burnside, Louisiana. It is interesting to note that all of these plants are located within the flood plain of the Mississippi River, lands subject to flooding before construction of the flood control works in the lower Mississippi River Valley. This is forceful testimony to the faith of industry in the flood control plan for this area.

But the developments in this area are by no means all along the lower Mississippi River. Throughout the area, an industrial complex is developing along this navigable waterway. Baton Rouge itself is often described as the most highly concentrated industrial site in the world. And the broad expanse of Mississippi Delta fields is broken often by a sprawling industry, manufacturing carpets, pharmaceuticals, fertilizer, or chemical products.

Future Development

It has been said that "What is past is prologue." In viewing the deep South's economic transformation, this statement seems particularly applicable. There is little doubt that the future holds great promise for this area because of its great water supply and water transportation network.

The lower Mississippi River, for example, provides an almost limitless supply of fresh water. At New Orleans, the annual mean flow of the Mississippi is at a rate in excess of 300 billion gallons daily, more than double the nation's daily usage for all purposes, including irrigation.

Reflecting the thoughts of industrialists, Mr. Dwight P. Joyce, Chairman and President of Glidden Company, says, "We intend to keep most of our future expansion on the water, because of the great potential there. We make extensive use of water transportation now for bringing in raw materials and shipping out our chemical products, and we believe that barge traffic will increase greatly in coming years."

CONCLUSION

With an abundance of water and a proven desire to expand, the deep South will continue to grow industrially. There will no doubt be demands for additional waterway improvements to link to this growing section other areas for an improvement of the nation as a whole. For the foreseeable future, the deep South holds great promise.

REFERENCES

1. "River Bank Boom," by Roger W. Benedict, Wall Street Journal, July 16, 1956.
2. Article on Intracoastal Waterway, Time, 1 October 1956.

FIGURE I. THE INLAND WATERWAY SYSTEM OF THE SOUTH

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998 (SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006 (HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3). 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU3), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

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JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(EM2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW1), 1245(HW1), 1246 (HW1), 1247(HW1), 1248(WW2), 1249(HW1), 1250(HW1), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW1), 1257(IR1)[c], 1258(HW1)[c], 1259(ST3)[c].

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c. Discussion of several papers, grouped by Divisions.

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